

Analysis of the impact of intermittent distribution by modelling the network-filling process

M. De Marchis, C. M. Fontanazza, G. Freni, G. La Loggia, E. Napoli and V. Notaro

ABSTRACT

In many countries, users acquire private tanks to reduce their vulnerability to water scarcity. In such conditions, water managers often apply intermittent distribution in order to reduce the water volumes supplied to the users. This practice modifies the hydraulic behaviour of the network and determines competition among users that need to collect enough water resource for their uses. Intermittent distribution is thus responsible for the inequality that can occur among users: those located in advantaged positions of the network are able to obtain water resources soon after the service period begins, while others have to wait much longer, after the network is full. This paper analyses the inequalities that take part when intermittent distribution is applied in water scarcity scenarios. Considering the complexity of the process, the analysis was performed by means of an unsteady numerical model. The model was applied to a real case study which provided interesting insights into the network filling process, helping to highlight the advantaged and disadvantaged areas of the network in different water scarcity scenarios.

Key words | intermittent distribution, pipe-filling process, private water tanks, water distribution network modelling

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INTRODUCTION

Water shortage is currently a major problem for social and economic growth in the world. Many recent programmatic reports (among others: [Intergovernmental Panel on Climate Change 2007](#)) foresee a drastic reduction of the water resources available for human purposes as a result of climate change. Although all of these studies show some uncertainty in quantifying the severity of this reduction, the unbalance between growing water demand and decreasing water resource availability is a real problem that should not be neglected.

Water shortage can be linked both to natural and climatic phenomena and human activities (such as incorrect management of supply networks, inefficient use of water resources, etc.). Thus, the negative effects of the severity and duration of

water shortage for people and economic activities depend on the climatic drought characteristics and the adaptability of human systems to the new environmental conditions.

In this context, distribution networks are usually the weak point of the whole water supply system. During periods of water shortage discontinuous water distribution and the rationing of water resources are methods often used to cope with water scarcity. Intermittent distribution, which is widely adopted in both developing ([McIntosh 1993](#); [McIntosh & Yñiguez 1997](#); [Hardoy et al. 2001](#); [Vairavamoorthy et al. 2001](#)) and developed countries ([Cubillo 2004, 2005](#)), has some advantages in that it requires little financial effort and reduces background water losses, however, it does not actually reduce user consumption, which is mainly a result of

users' habits and socio-economic level. Moreover, it leads to network operating conditions that are not accounted for in the typical design (Fontanazza *et al.* 2007, 2008).

The water utility company tries to distribute limited water resources as efficiently as possible, dividing the entire network into different zones defined by the number of users, and supplying each zone with a ration of the available volume for fixed periods of time (usually less than 24 hours). Users try to cope with water service intermittency by using private tanks (Figure 1).

The tanks are located between the revenue water meter and the users' property, usually on rooftops, and are often over-designed to take into account possible higher water consumption and leakages (Arregui *et al.* 2006; Cobacho *et al.* 2008; Criminisi *et al.* 2009). Therefore, node water

demand does not depend on actual user consumption, but rather on the node water head.

In this complex situation, network behaviour and the accessibility of water resources to users are related to pressure levels – advanced tools are needed for analysing and representing the system's ability to meet users' needs.

Since the 1980s, researchers have proposed various methods to compute actual water consumption, node pressures and flows in networks operating in conditions different from design ones (such as intermittent systems). Most of the proposed methods involve an assumption on the relationship between pressure and outflow at the demand nodes. These methods are generally termed head-driven approaches. Bhave (1981), who first acknowledged that demand-driven analysis does not behave well when node

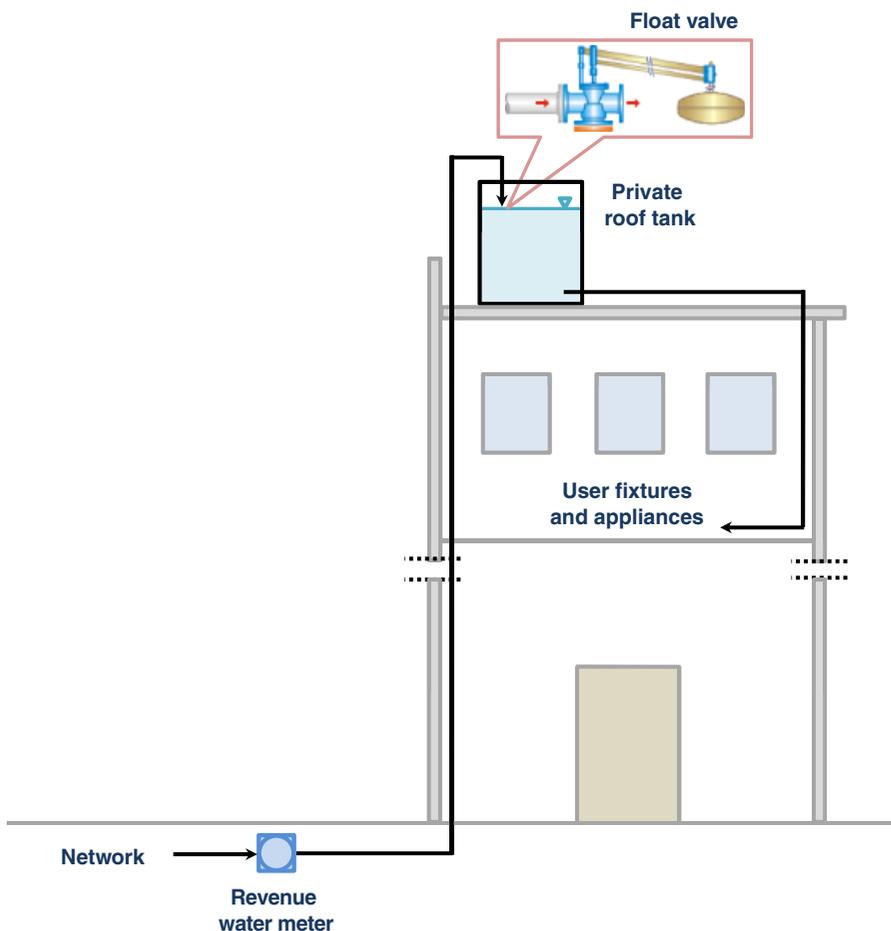


Figure 1 | A schematic of a typical plumbing connection to a private roof tank.

heads are lower than required service standard ones, proposed a pressure–consumption relationship where the actual outflow at the node is equal to zero when the available water head is lower than the minimum required to have outflow and it is equal to the required outflow otherwise. Germanopoulos (1985) suggested the use of an empirical pressure–consumption relationship to predict outflows at various nodal head and leakage levels considering flows and head simultaneously. Then, Wagner *et al.* (1988) and Chandapillai (1991) proposed the use of a parabolic curve to represent the pressure–consumption relationship at a demand node for head between the minimum required to have outflow at the node and the head required to satisfy the water demand. In these models the outlets are controlled because when the head is higher than required, the outflow is constant and equal to the demand. Reddy & Elango (1989) introduced a method completely different from the others previously referred to. Reddy & Elango suggested a pressure–consumption function without an upper boundary: when the head is higher than minimum, the node outflow increases as a power function; otherwise it is equal to zero. Later, Gupta & Bhawe (1996) reviewed different methods concluding that the method using parabolic head–discharge relationship is the best. However, Ackley *et al.* (2001) pointed out that the proposed pressure–demand models do not satisfy the requirements of being hydraulically meaningful (as Wagner’s one) and at the same time smooth and differentiable. Tucciarelli *et al.* (1999), Tanyimboh *et al.* (2001) and Tanyimboh & Templeman (2004) presented a modification to Wagner *et al.*’s 1988 model in order to assure its differentiability; Todini (2003) introduced the pressure-driven scheme within a global gradient algorithm (GGA). Ang & Jowitt (2006) presented a different approach termed the pressure-deficient network algorithm to solve and provide additional insight into the behaviour of water distribution networks operating under pressure-deficient conditions. Giustolisi *et al.* (2008a, b) introduced an over-relaxation parameter and used Germanopoulos’ (for background leakages) and Wagner’s (for demands) models in pressure-driven analysis using GGA. Piller & Van Zyl (2007, 2009) contributed to pressure-deficient network analysis proposing the use of Collins *et al.* (1978) ‘content’ and ‘co-content’ models within the pressure-driven framework.

In pressure-driven analysis, as well as in demand-driven analysis, total demand along the pipe is represented as two lumped withdrawals at its terminal sections. Some recent findings show that this simplification could generate head-loss errors in steady-state simulation of networks. Giustolisi & Todini (2009) proposed a general framework for uniform demand distribution in demand-driven analysis and Giustolisi *et al.* (2009) for a generic connection pattern in pressure-driven analysis. Finally, Giustolisi (2010) presented an extension of the GGA permitting the effective introduction of the lumped nodal demands while preserving the energy balance by means of a pipe hydraulic resistance correction. Nevertheless, the simplification of the demand distribution as concentrated withdrawals is still in use due to a lack of knowledge about the number and the position of actual user connections.

An analysis of the literature left some questions unanswered, especially with regard to node demand formulation, which is usually not physically based, and the simulation of the network’s filling and emptying phases, which are generally neglected when adopting permanent flow conditions.

A model that accounts for the unsteady filling process was proposed by Liou & Hunt (1996); the model results were in good agreement with experimental measures but the model was only used for relatively simple pipeline systems and it is not suitable for complex water distribution networks.

The present paper proposes an unsteady model for analysing the filling process of water networks that takes part with intermittent distribution and water shortage. The model is based on the hypotheses that atmospheric pressure is maintained in network pipes when they are empty and that the water column can not be fragmented. The network model is integrated with a node demand model based on the node pressure–consumption law, which defines flow drawn from the network for filling users’ tank. The proposed model was calibrated on a real case study, the distribution network of Palermo (Italy), was monitored during both continuous and intermittent distribution. The model was used for investigating the inequalities among users that are determined by intermittent distribution in water scarcity conditions. After testing its reliability the model was also tested against real data that can be used by the water managers in order to prevent and mitigate the detrimental effects of intermittent distribution or for proposing alternative management policies.

THE NUMERICAL MODEL OF THE NETWORK FILLING PROCESS

During a shortage period, the water resource distribution in the network can be governed through the use of gate valves – the opening or closing of which allows the choice of which part of the network should be full or empty. With the aim of analysing, through numerical modelling, the network filling process, it was assumed that all of the pipes in the network are initially empty, these empty pipelines are connected to the network reservoirs and the filling of the network starts after the opening of the gates (Figure 2(a)). This hypothesis (as discussed further below) is corroborated by the fact that, in intermittent supply, users activate local pumping stations at the end of the service period for collecting residual water resources in the network. At the beginning of the filling process, in order to impose the initial boundary condition in the inlet section, it is assumed that the pipe connected with the reservoir is slightly full of fluid (for a length of 1 cm). This is a numerical condition needed to apply the method of the characteristic. The initial velocity of the water front inside a previously empty pipe can be quite high since the pressure gradient is relatively high due to the rapid change in pressure, which, as specified in the following, can be considered atmospheric at the water front (Figure 2(a)). When the piezometric gradient between the inlet and outlet sections of the pipe is relatively low, as it is in the network analysed here, as the

length of the water column increases, the pressure gradient reduces, and the frictional water losses are increased proportional to the length of the column. The increase of frictional losses can progressively reduce the velocity of the water front. As the water front reaches one of these users' connections, tanks start to fill (Figure 2(b)), with a discharge depending on the geometric and hydraulic features as well as on the pressure at the derivation point. When the water front reaches the end of a pipeline (Figure 2(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. In real water distribution networks a pipeline could be filled from both ends (Figure 2(d)). The two water fronts proceed inside the pipe one against the other. Once they reach the same cross section, the subsequent collision can cause an increase in pressure, which again travels through the network pipelines with high celerity. The increase of the water head due to the collision of water fronts can be observed from the analysis of the time series of the simulated water head. Due to the hypothesis that the air pressure at the water front face is always atmospheric, in this case the increase of the pressure of the air entrapped between the two opposite water fronts is not considered.

Due to the extreme complexity of the multi-phase phenomena that occurs during the filling of the network, a number of simplifying assumptions must be used.

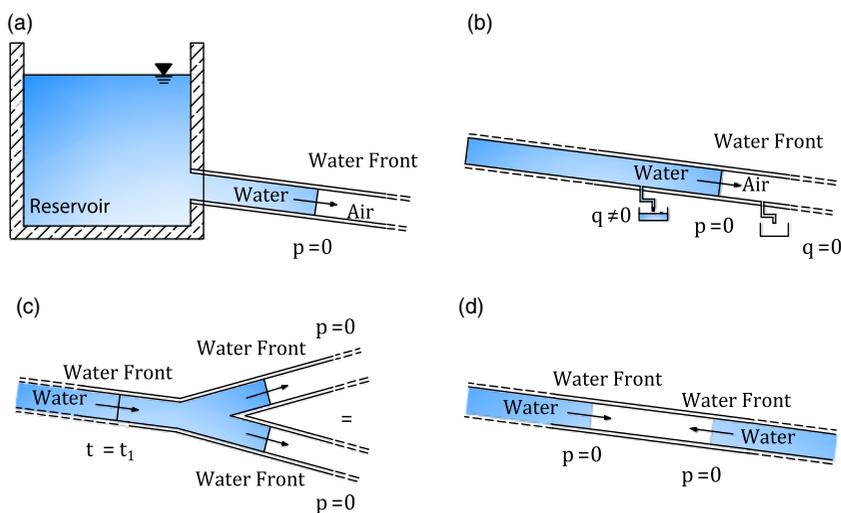


Figure 2 | Hydraulic schematics of the network filling process: a) initial phase of water front propagation; b) water front reaches a user connection; c) water front reaches the end of the pipeline; d) water fronts proceed in opposite directions.

Specifically, following Liou & Hunt (1996), it is assumed that the air pressure at the water front face is always atmospheric, the water column fronts coincide with pipeline cross sections (thus, every cross section is entirely wet or dry) and the steady-state formula can be used to calculate frictional resistance. As demonstrated by De Marchis et al. (2009), the three hypotheses can be considered reasonably correct.

Liou & Hunt (1996) used the additional hypothesis of rigid water-column, which can be accepted for simple pipeline systems like those considered in their research, but is highly questionable in complex water distribution networks. Previous research carried out using the rigid water-column approach clearly showed the occurrence of exceedingly high unphysical pressure peaks during the filling of the network (Curto et al. 2007).

In this paper, the method of characteristics (MOC) is used to simulate the filling process. The MOC approach reduces the momentum and continuity partial differential equations for weakly compressible fluids flowing inside deformable pipelines into ordinary differential equations, which can be solved through finite-difference methods. These equations, known as compatibility equations, can be written as follows.

$$\frac{dV}{dt} + \frac{g}{c} \frac{dh}{dt} + gJ + Vsen\theta = 0 \quad (1)$$

$$\frac{dV}{dt} - \frac{g}{c} \frac{dh}{dt} + gJ - Vsen\theta = 0 \quad (2)$$

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; j is the head loss per unit length of pipe according to the Darcy–Weisbach equation; and θ is the slope of the pipeline. Equations (1) and (2) are valid only along their characteristic curves, $ds/dt = +c$ and $ds/dt = -c$, respectively. The iterative solution of Equations (1) and (2) results in a space and time dependent distribution of velocity and water head (De Marchis et al. 2009).

To reduce system complexity, it is assumed that water supply occurs only at nodes where two or more pipelines cross. This is one of the typical skeletonizing assumptions made for maximizing the efficiency of water distribution network numerical models. The integration time step must be selected according to the following condition: $\Delta t_i = \frac{L_i^n}{c_i N_i}$,

where L_i^n and C_i are the length of the water column and celerity of the i -th pipe at the time t^n , respectively. The celerity of the i -th pipe is calculated taking into account the compressibility of the fluid and of the elasticity of the walls of the pipe $c_i = \frac{c}{\sqrt{1 + \epsilon}}$ with ϵ the coefficient of the compressibility of the fluid assumed to be equal to 10^9 N/m², D_i is the i -th pipe diameter, e is the thickness and E is the modulus of elasticity of the pipe. The modulus E was assumed to be 10^9 N/m² due to the specific type of pipes in the network considered. For pipes characterized by a thickness of about 2 cm, like those considered in the present analysis, the celerity ranges between 755 and 1135 m/s depending on the pipe diameters considered.

The number of sections N_i in which the i -th pipe is subdivided is calculated dynamically during the filling process. Specifically, $N_i = 10$ for empty pipes, while its value is proportionally reduced with the length of the pipe filled. The number of ten segments has been chosen in such a way to reduce the time cost of the simulation and to achieve a good approximation of the solution of the MOC.

In order to resolve the compatibility equation in the whole network, there is the need to choose a unique time step of integration. After the calculation of the time step for each pipe, the minimum value is calculated according to $\Delta t = \min_i$ and imposed as the unique time step for the advancement of the simulation. The imposition of the minimum time step Δt requires the need to interpolate the values of the water head and of the velocity along the new characteristic line (for details on interpolation see De Marchis et al. 2009).

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. Figure 3 shows a schematic of the modelled elements with the indication of the main variables.

Each network node supplies as many tanks as buildings considered connected to it. Specifically, the discharge entering the k -th tank connected to the j -th node $q_{up,k}^j$ can be obtained as:

$$q_{up,k}^j = C_v \cdot a \cdot \sqrt{2gP_k^j} \quad k = 1, \dots, T \quad (3)$$

where C_v is the non-dimensional float valve emitter coefficient; a is the valve effective discharge area; P_k^j is the

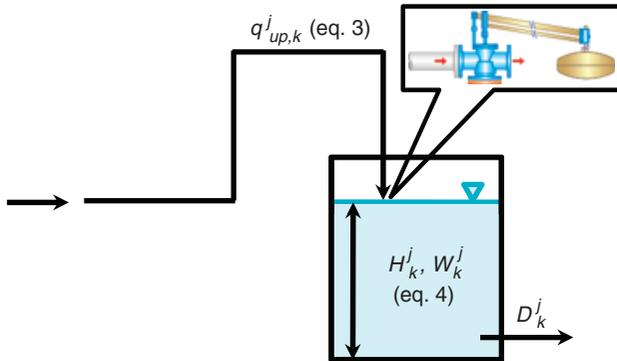


Figure 3 | A schematic of the modelled system.

hydraulic head over the k -th private tank; g is the gravity acceleration; and T is the number of tanks connected to the node. The coefficients C_v and a were experimentally obtained by field studies on the analysed network and are further described in Criminisi et al. (2009). Equation (3) can be used to calculate the discharge at the j -th node entering the k -th tank only when the floating valve is open, that is, before the user tank is entirely filled. Thus, this equation must be combined with the tank continuity equation, which can be written for the j -th node as:

$$\begin{cases} q_{up,k}^j - D_k^j = \frac{dW_k^j}{dt} = A_k^j \frac{dH_k^j}{dt} & \text{for } H_k^j < H_{max,k}^j \\ q_{up,k}^j = 0 & \text{for } H_k^j \geq H_{max,k}^j \end{cases} \quad (4)$$

where D_k^j is the user water demand; W_k^j is the volume of the k -th storage tank having area A_k^j and variable water depth H_k^j ; and $H_{max,k}^j$ is the maximum allowed water level in the tank (before the floating valve closes).

Finally, the discharge, Q_{up}^j , and the water demand of users considered lumped at the j -th node D^j are:

$$Q_{up}^j = \sum_{k=1}^T q_{up,k}^j \quad (5)$$

$$D^j = \sum_{k=1}^T D_k^j \quad (6)$$

The solution of the compatibility Equations (1) and (2) with the proper boundary conditions results in the velocity V and the water head h for each time step in the selected cross sections of the pipelines. The filling process is then updated,

calculating the length L_i^{n+1} of the water column inside the partially empty i -th pipeline as:

$$L_i^{n+1} = L_i^n + V_N^{i,n+1} \Delta t \quad (7)$$

where V_N is the velocity of the water front face of the pipeline.

When the length of the water column achieves the length of the whole pipeline, the update stops while the connected pipes start to fill according to Equation (7). The filling process continues until all pipelines are completely filled. At the same time, the water levels inside the user tanks are updated according to Equation (4). The parameters of the node demand formulation Equations (3) and (4) have to be calibrated experimentally as discussed below. After the network filling process, the computations are carried out until the steady-state condition is achieved in all nodes of the distribution system.

THE CASE STUDY

The proposed model was applied to one of 17 supply networks of Palermo, Italy (Figure 4).

All of the geometric characteristics of the network as well as the number and distribution of user connections, the water volumes supplied and measured, and the pressure and flow values in a few important nodes are precisely known. The network was, in fact, totally rebuilt in 2002 even though the old cast-iron feeding pipes still remain in service. These pipes connect the network with two reservoirs at different levels that can store up to 40,000 m³ per day and supply the 8,700 user connections of the network (nearly 35,000 inhabitants).

The entire network is made of polyethylene and it is about 40 km long. The pipes have diameters ranging between 110 and 225 mm, the network node (street-level user connections) elevation ranges from 3 to 47 m above sea level, while building height ranges from 5 to 50 m. The network was designed to supply about 400 l/capita/day, but actual average consumption is about 260 l/capita/day. As a consequence, the network was characterized by low water velocities and correspondently high pressures, which resulted in high leakage levels (around 30 per cent of the network inflow volume) in the past mainly due to pressure cycles between day (with

pressures on the network nodes in the range between 2 and 4 atm) and night (with pressures reaching 8 atm in the lower part of the network). In order to reduce the impact of leakages in water scarcity conditions, intermittent distribution on a daily basis was introduced as a common practice over the last five years (at least during summer period), leading users to acquire and install private tanks. Such a practice had the main disadvantage of generating inequity among users in terms of water supply and this aspect was the main objective of the present modelling study. Several other negative impacts were reported both regarding the water service quality and the durability of pipes, such as water quality deterioration, the risk for pollutant intrusion in the empty pipes and deterioration of the network (Andey & Kelkar 2009). Such other aspects were not considered in the present study as that was focused on the unequal distribution of water resources among users.

Initially, the water utility company carried out a field campaign in order to investigate the characteristics and the geometry of such private tanks (Criminisi et al. 2009). In the analysed network, the tanks are all located on the roof-tops of the buildings, they are usually modular, that is, made of several small interconnected tanks, having a maximum water depth of between 1 and 1.5 m and specific volume equal to 200–250 l per connected inhabitant. The field campaign showed that such private storage systems are often supplied by local pumping systems directly connected to the network that is used to supply private tanks when network pressure is not sufficient. Such pumping systems are usually manually operated by the users at the end of the service period, when the network is emptying, in order to collect the residual water volumes contained in the network pipes. Figure 5 shows the node elevation and the maximum height of the tanks connected to each network node. Data about

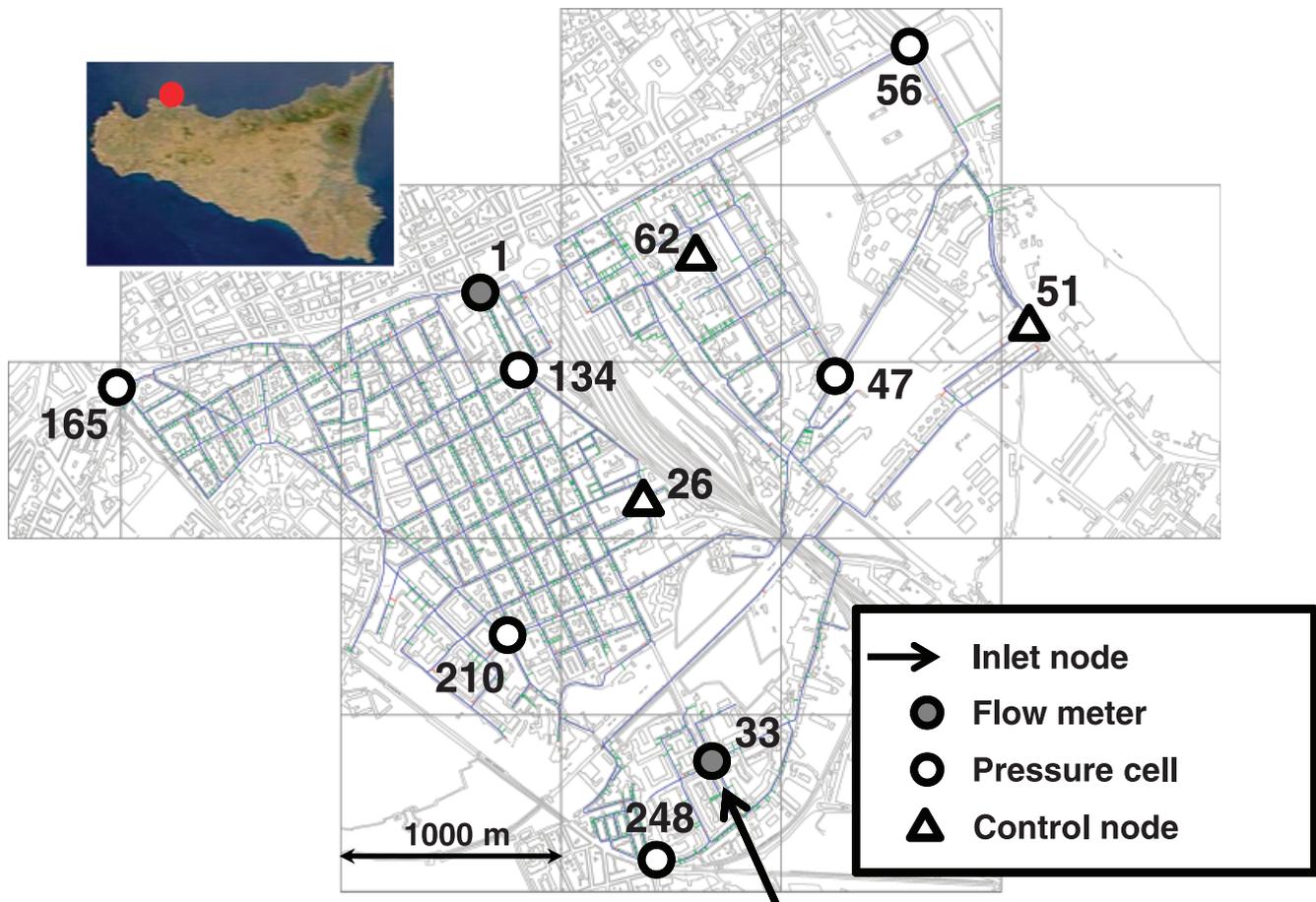


Figure 4 | A schematic of the network.

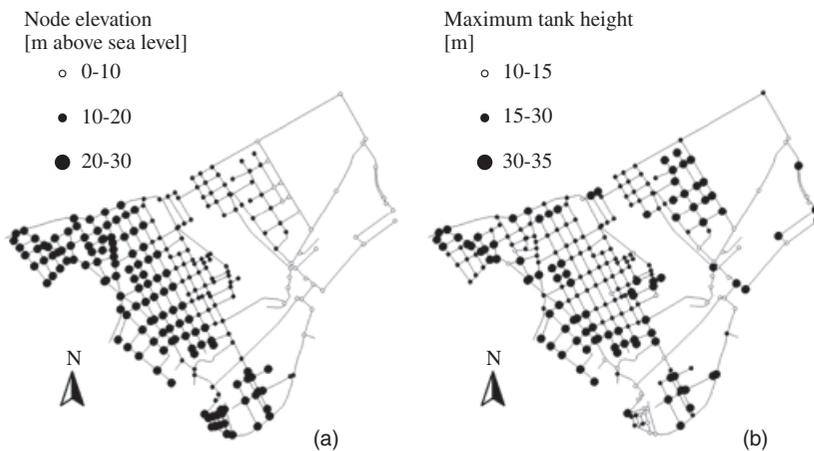


Figure 5 | A map of the network node elevation (a) and the maximum height of the tanks connected to each node (b).

building heights as well as inhabitants were collected during the field survey in collaboration with Palermo municipality.

The network is monitored by six pressure cells and two electromagnetic flow meters (Figure 4) that have provided data on an hourly basis almost continuously since 2001. The calibration of the network hydraulic model is constantly updated when new data become available (Fontanazza *et al.* 2007, 2008). In this paper, the pressure data used to represent the filling and the emptying processes have a time resolution of five minutes and refer to the period between June and October 2002, during which the network was managed by intermittent supply on a daily basis. For the same period, flow data entering the network were available with the same temporal resolution. During the monitoring period, per capita water supply was not reduced and intermittent supply was mainly adopted for reducing network leakages.

MODEL APPLICATION

In the present application, the network was schematized by approximately 400 pipes and 270 nodes. Users were aggregated to the network nodes according to the following simplifications.

- Each building was connected to the node of the network nearest to the real user connection.
- For each building, a single tank was considered positioned on the roof-top and having a maximum water depth equal to 1.5 m and volume equal to 200 l per the number of the inhabitants of the building.

- Local pumping systems were not simulated as they are manually operated and they are usually not working during the initial network filling process.
- The node head-discharge model parameters were set to the averages obtained in a field campaign that was carried out in the same network in 2007 and 2008 (Criminisi *et al.* 2009): C_v is set equal to 0.57 and a to 2.8 cm².

For the sake of simplification in the following model application, the average network pressure profile and inflow patterns are computed for the 78 daily time series for which both hourly pressures and discharges were available.

RESULTS ANALYSIS

The model was initially calibrated according to the pressure profiles available during the monitoring period in the six pressure gauges of the network (Figure 4). After calibration, the model was used for evaluating the inequalities among users even if water availability is equal to water demand. Inequality increases if water scarcity scenarios are considered; in the present study, three hypothetical scenarios were simulated by progressively reducing water supply from 100 to 90 per cent, 70 and 50 per cent of users' water demand.

Model calibration

Figure 6 shows the agreement between the simulated and measured pressure time series.

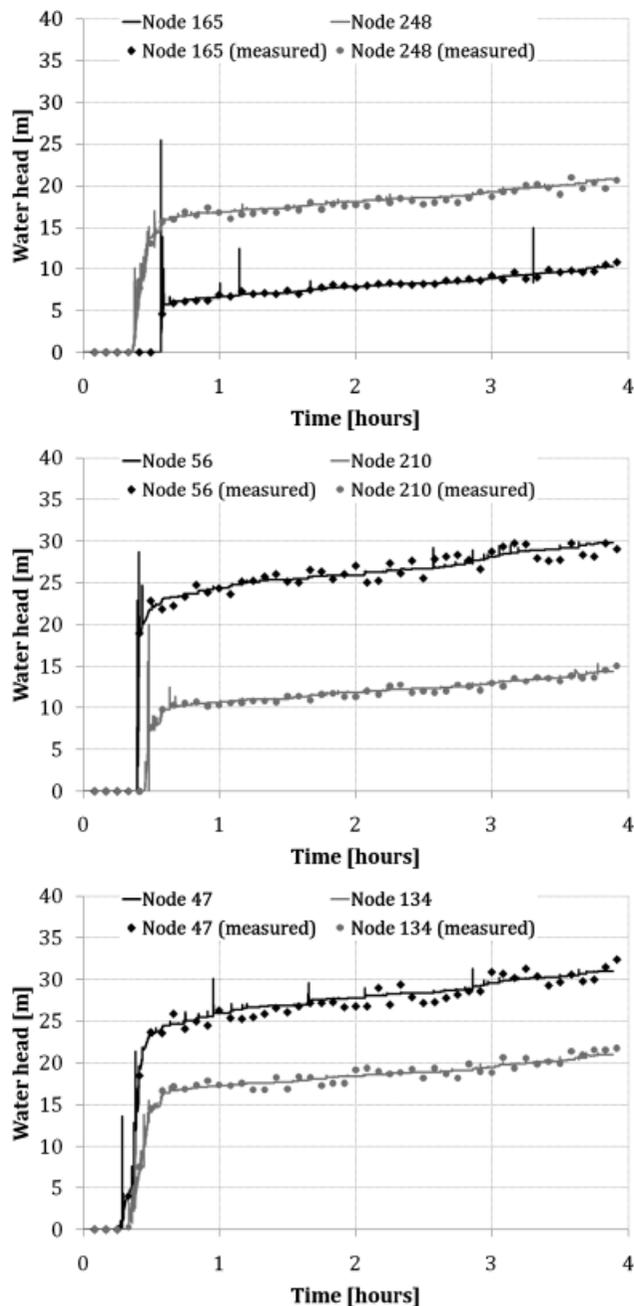


Figure 6 | Comparison between simulated and measured pressures in the gauge nodes (24/06/2002).

The peaks in simulated pressures result from small water hammer events due to the opening and closing of the ball valves supplying the private tanks near the pressure gauges and to the collision between the water fronts, demonstrating the sensitivity of the proposed modelling approach.

In each scenario, the filling process is adequately simulated by the model: initially, each monitored node receives water depending on its distance from the inlet node; the pressure rises rapidly until private tanks start to be supplied depending on their height above ground (usually between 5 and 50 m). Then, pressure rises gradually as private tanks become full.

Analysis of network filling when water demand is fully satisfied

The results of the filling process analysis of network show the high inequality in supplying users even in the condition when 100 per cent of water demand is delivered.

Figure 7 shows the dynamics of pipe filling and clearly indicates the location of the disadvantaged users in the left corner of the network.

Network mains start filling very soon after the water network is supplied. After 600 seconds, only a few secondary pipes near the inlet node are filled, while the upper part of the network is still empty after 1800 seconds. The network is entirely filled about one hour after water supply begins, but the pressure levels for several users are still inadequate to allow for private tank supply. The high consumption of the advantaged users due to the filling of their private tanks at the beginning of service protracts the time needed to fill the network completely and generates high head losses in pipes, conveying discharges higher than the design node demand.

The likelihood of private tanks being filled depends on the water head of building roof tanks, so all of the advantaged users are not necessarily in the topographically lower part of the network due to the combination of average building height and pressure level. Only a few users can be supplied by the network after the filling process finishes (after 3950 s) because pressure levels on the network are not sufficient to supply the private tanks.

As Figure 8 shows, the pressure level is adequate to supply only a few private tanks up to three hours after the filling process starts. The number of nodes supplied increases with the time, even if, twelve hours after the filling process starts, the pressure in the network is lower than the building average height in some nodes. Due to low pressure (less than about 1 atm), the disadvantaged users are not supplied until



Figure 7 | Average dynamics of pipe filling in time and space.

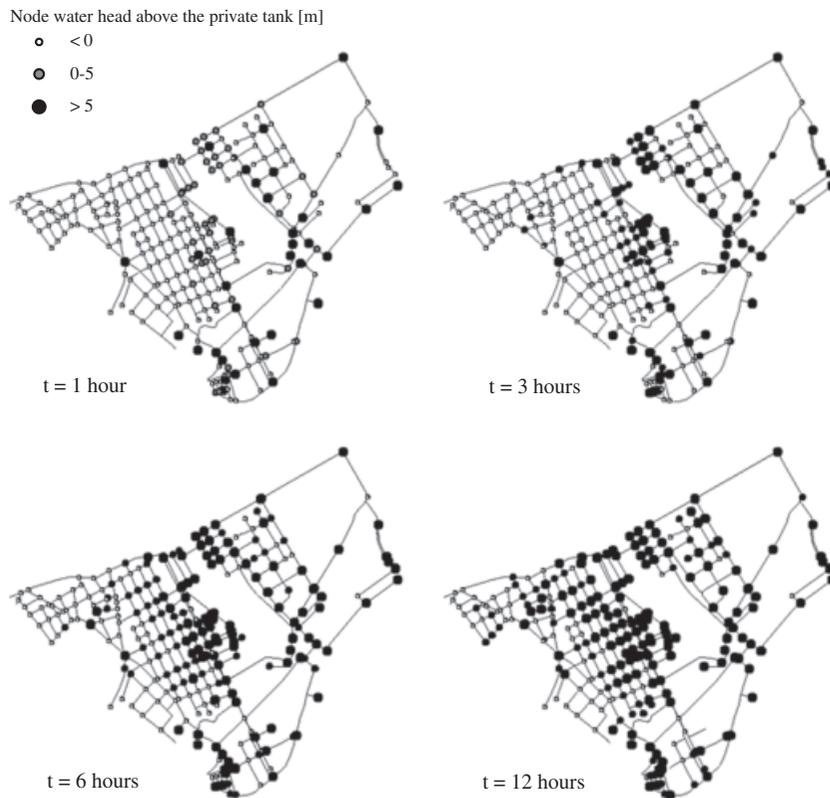


Figure 8 | Temporal pattern of the node water head over the private tank.

much later. All the users are supplied within the service day and the inequalities among them are limited to the timing: advantaged nodes are supplied immediately and disadvantaged nodes have to wait till the end of the service period.

This situation is well documented at network scale in Figure 9, where the time needed to begin (Figure 9(a)) and complete (Figure 9(b)) the filling of the local tanks for the highest users in each node is reported. A few users' tanks are completely filled within 5 hours, while others have to wait up to 8 hours for the tank-filling process to start, confirming that intermittent distribution can increase users' vulnerability to water shortage. As shown in Figure 9(b), the most disadvantaged users are still able to collect all the water they need during the daily intermittent service considered in the present study, but they would have to compress their demand if service were reduced to less than 20 hours.

Analysis of water scarcity scenarios

The inequalities increase if a water scarcity scenario is considered: if the water volume supplied is lower than the water demand, iniquitous distribution among users is not limited to the service timing. Some users, in fact, receive their entire water demand, others only a small amount. In order to better explain these inequalities, two synthetic indicators WV_t^j and $WV_{global,t}$ were proposed.

The first indicator WV_t^j is calculated for the j -th node according to the following equation.

$$WV_t^j = \frac{\int_0^t Q_{up}^j(t) dt}{\int_0^{24} D^j(t) dt} \quad j = 1, \dots, M \tag{8}$$

where the symbols have the meaning described in the previous paragraphs, M is the number of network nodes and the integration is over time t . The indicator expresses the ratio between water volumes supplied over time and daily water demand of the users connected to the j -th node.

The second indicator $WV_{global,t}$ is a weighted average used as global system parameter, obtained according to the following equation.

$$WV_{global,t} = \sum_{j=1}^N (WV_t^j \cdot p^j) \tag{9}$$

where p^j is the weight linked to the j -th node and calculated as the ratio between the daily water demand of the users connected to the j -th node and the users daily water demand of the whole network.

With regard to the first indicator WV_t^j , in normal conditions (sufficient water supply and pressure to feed all users, without the need of tanks), WV_t^j grows progressively during the day, following users' water demand and is equal to one at

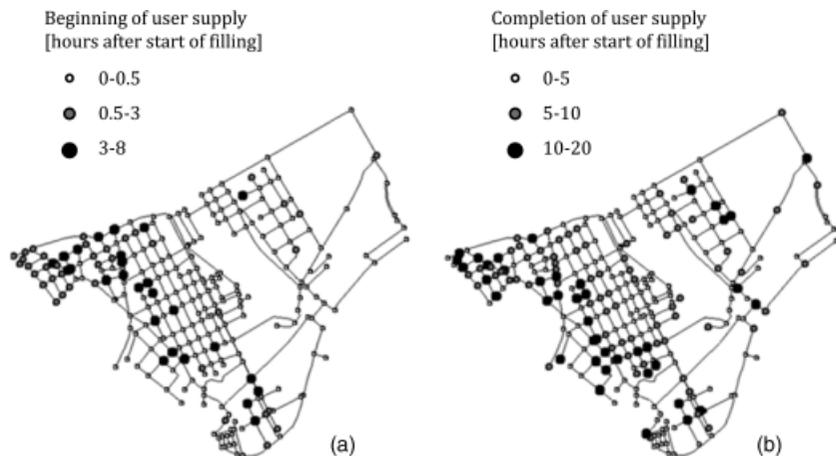


Figure 9 | Time needed to begin (a) and complete (b) the filling of the private tanks.

the end of the day. In water scarcity conditions, it will always be lower than one for all those disadvantaged users that are not able to satisfy their water demand; the indicator reaches the unit rapidly for those users that are in advantaged positions in the network. Figure 10 shows the temporal variation of the indicator in three of the control nodes indicated in Figure 4. In the figure the four scenarios of water availability, considering the presence of the tanks, are compared with the normal scenario, characterized by sufficient water supply and network pressures for complying with users needs (no tanks). The figure shows the great impact of the tanks that can be measured by the distance between the normal scenario and any other: in the normal scenario, the water volume supplied to the node depends on users demand; in the other scenarios, independent from water availability, the water volume supplied to the node is only connected to tank filling process than is controlled by the network pressure.

Comparing different scenarios involving storage tanks, the following considerations may be derived.

- Figure 10(a) shows a typical advantaged node that in any water scarcity scenario is able to rapidly collect the whole water volume needed for filling the tanks. For this node, the difference between water scarcity scenarios and complete water supply is irrelevant and it is only related to tank filling time ranging between less than one hour and more than two hours.
- Figure 10(b) shows an intermediate node that can be advantaged or disadvantaged depending on water scarcity scenario. In the case of 100 per cent water availability, tank filling starts a few hours after the beginning of the service day and finishes in about 12 hours. If the network is affected by a mild water scarcity scenario (90 per cent water availability), this node is almost unaffected being able to collect 100 per cent of users' water demand. Even when water availability is reduced to 70 per cent, the node can collect 80 per cent of users water demand, suffering water scarcity less than the network average. In the case of severe water scarcity (50 per cent water availability), the node is subjected to heavier conditions than the average, being able to collect only 30 per cent of users' water demand.
- Node 26 (Figure 10(c)) is a typical disadvantaged node in which tank filling time is higher than in the others nodes.

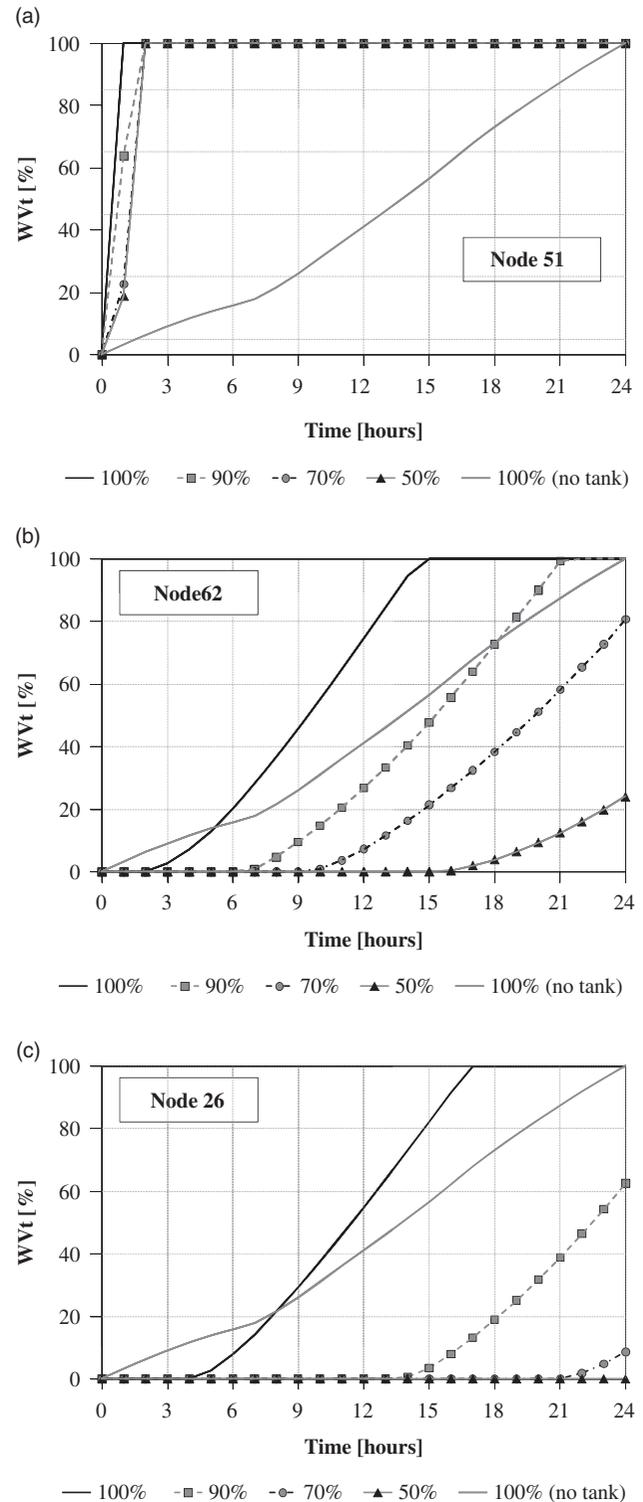


Figure 10 | WWt in some control nodes in case of 100% water availability (with or without storage tanks) and in different water scarcity scenarios (water volumes availability being 90, 70 and 50% of users' water demand).

It always suffers water scarcity conditions heavier than the network average: in severe conditions, the node does not receive water volumes from the network.

The condition of being advantaged or disadvantaged in the network cannot be stated by a simple analysis on the node elevation or on the node position in the network. Rather, a combination of these factors may be determinant on the impact of water scarcity on the users and the use of unsteady models can be a powerful analysis tool. These considerations are confirmed by Figures 11 and 12 in which WVt is shown at network scale for $t = 12$ hours and $t = 24$ hours (in the middle and at the end of the service day) and for 100, 90, 70 and 50 per cent water availability scenarios.

Figure 11 shows that a few nodes (less than 10 per cent) can collect their entire water demand soon and well before the end of the service day in any water availability scenario; these nodes are the first to be served and the first filling their

tanks. The comparison of the scenarios at the end of the service day (Figure 12) shows that almost 20 per cent of the nodes are in the advantaged condition, not being affected by any of the analysed water scarcity scenarios. Around 50 per cent of the users are in the intermediate condition, being advantaged in mild and average scarcity condition but becoming disadvantaged in the severe scenario. The remaining 30 per cent of the users are in a disadvantaged condition; they are distributed over the network even if they are more likely to be in the upper part.

The relevant negative impacts of intermittent supply in terms of inequality and competition generated among users is confirmed by the results obtained for the global system indicator $WV_{global,t}$ (Table 1). Even if the network is affected by a mild water scarcity scenario (90 per cent water availability), the network is able to supply only the 53 per cent of the demand of the whole network after 12 hours of the beginning of the service and only the 75 per cent after

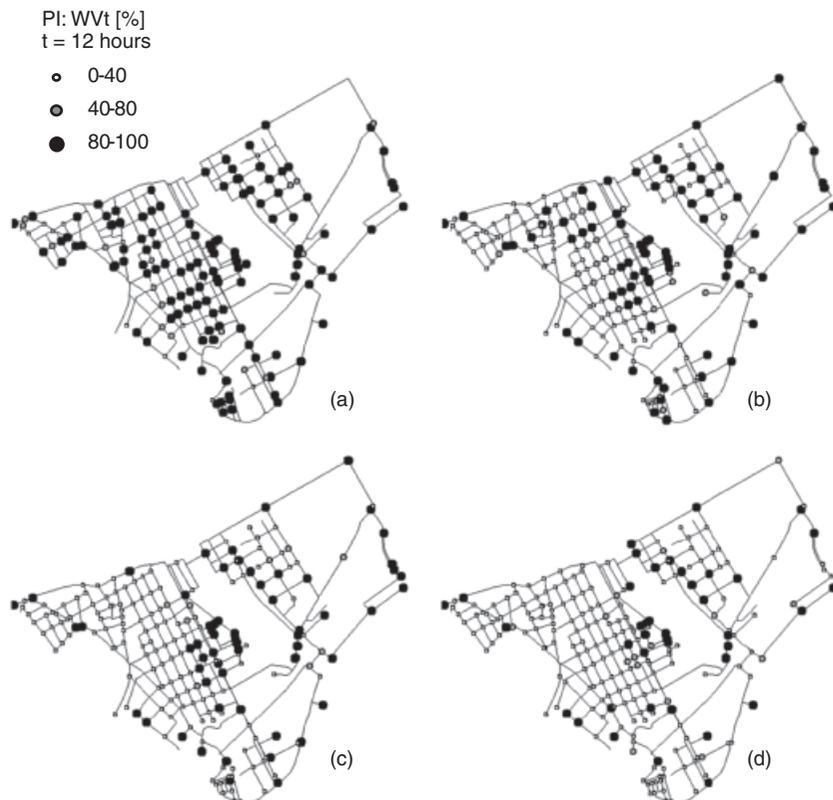


Figure 11 | Distribution of WVt over the network after 12 hours from the beginning of the service day: a) 100% water availability (with storage tanks) and water availability being 90% (b), 70% (c) and 50% (d) of users' water demand.

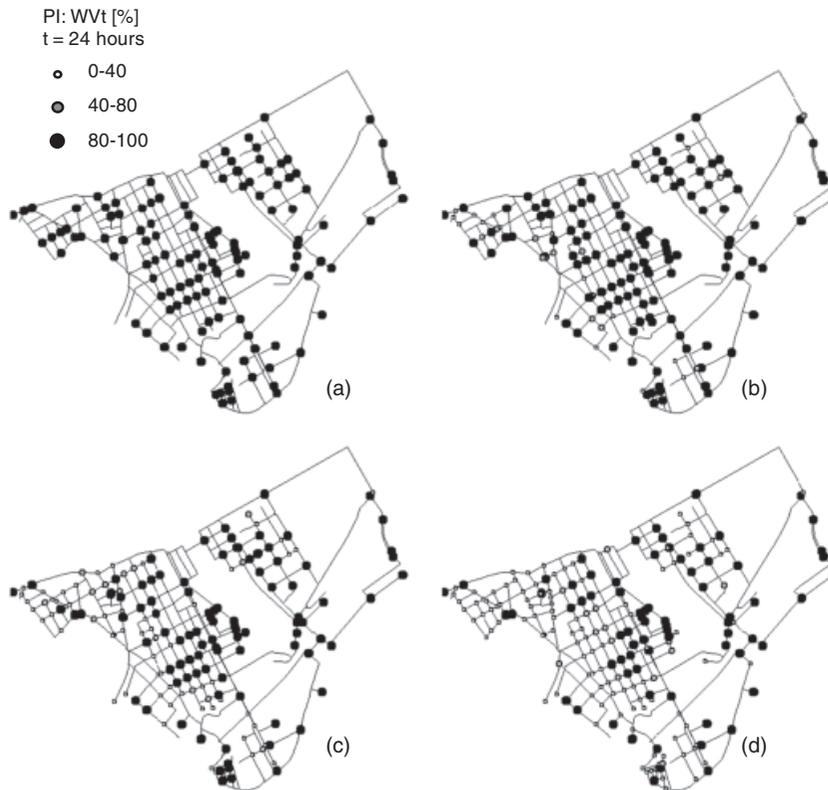


Figure 12 | Distribution of WVt over the network at the end of the service day: a) 100% water availability (with storage tanks) and water availability being 90% (b), 70% (c) and 50% (d) of users' water demand.

18 hours. In the average water scarcity scenario (70 per cent water availability) only the 38 per cent of the user demand is supplied after 12 hours, and only 56 per cent after 18 hours.

CONCLUSION

The present paper discussed the development and application of a numerical model for simulating the filling process of a distribution network managed via intermittent supply.

Table 1 | Temporal pattern of the global system indicator $WV_{global,t}$ in different water availability scenarios

Time (h)	$WV_{global,t}$ (%)			
	scenario 100%	scenario 90%	scenario 70%	scenario 50%
6	58	24	16	10
12	90	53	38	25
18	99	75	56	39
24	100	90	70	50

The problem is made complex by the presence of the tanks acquired by the users in an attempt to reduce their vulnerability to service intermittency.

The tanks greatly affect the hydraulic behaviour of the network modifying the demand pattern of the users. User demand is in fact much higher than normal at the beginning of the service period reducing the pressure level on the network and presenting some disadvantaged users to be supplied.

This paper demonstrated the reliability of the proposed model by means of application to a real case study. The proposed approach is even more relevant when water scarcity scenarios are analysed allowing for identifying advantaged and disadvantaged users.

The application of the model confirmed the relevant negative impacts of intermittent supply, allowing some users to collect their entire daily water demand in few hours, and the inequality and competition that intermittent supply generates among users. Analysis results may be summarized in the following points.

- Advantaged nodes start receiving water only a few minutes after the beginning of network filling; the pipes connected to the disadvantaged nodes start to fill after more than 30 minutes. Unfortunately, due to low pressure, the most disadvantaged users do not receive water until much later (about 7 hours).
- The daily supply of disadvantaged users is not complete until after 20 hours, thus confirming their high vulnerability to intermittent distribution. In the analysed case, daily intermittent distribution allows all users to collect the required volume, but if the service time were shorter than 20 hours, the water demand of some disadvantaged users would not be met, generating inequalities.
- The network pressure is generally low for the most part of the service day because of the presence of the tanks and it increases slightly along the day.
- In water scarcity scenarios, advantaged users are not affected, being able to collect 100 per cent of users' demand, while some disadvantaged users collect much less water than the network average.
- The modelling application showed that some nodes (50 per cent of the total in the present application) can be advantaged or disadvantaged depending on the severity of the water scarcity scenario.

This result may be useful for evaluating the most efficient strategies for managing the network in order to reduce high inequality due to intermittency in the water supply service. Future research could aim to apply the model for evaluating network management practices, such as Pressure Management Areas, that are available as alternatives to intermittent distribution; these practices may help solve the problem of network management under water scarcity without the negative effects that result from service intermittency.

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