A mathematical model to evaluate apparent losses due to meter under-registration in intermittent water distribution networks

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

ABSTRACT

Apparent losses consist of water volume drawn from the network, consumed by users but not paid for. Those due to water meter under-registration were evaluated by means of a mathematical model able to analyse the complexity of intermittent supply systems with private tanks. This supply scheme is very common in the Mediterranean area where unexpected water shortage often happens and intermittent water supply is a common practice. In order to analyse such complex systems, a demand model, reproducing the effect of private tanks, and an apparent losses module were developed and coupled with an hydraulic network model based on the Global Gradient Algorithm (GGA). In distribution networks pressure reduction valves (PRVs) are often used by water utilities to control the pressure and reduce background losses. These practices could influence the performance of water meters. For this reason, a PRV model was implemented and integrated with the demand and the hydraulic network models to better estimate the effect of pressure management on real and apparent losses. The comprehensive model was applied to a real case study. The proposed modelling approach was used to identify regions of the network with high apparent losses. Furthermore, the model may be used to predict the results of a water meter replacement plan and of the installation of devices that could affect apparent losses.

Key words: apparent losses, private tank, water meter substitution plan, water meter under-registration

INTRODUCTION

Water losses can be divided into real and apparent losses according to the International Water Association (IWA) water balance (Lambert & Hirner 2000; Lambert 2002). Real losses are the annual volumes lost through all types of leaks, bursts and overflows in mains, service tanks and service connections, up to the point of customer metering. Apparent losses consist of unauthorised consumption and all types of metering inaccuracies (Lambert 2002). Apparent losses also include the non-physical financial losses related to water that is being consumed and metered but not paid for. Until now, procedures and guidelines for the assessment and the control of apparent losses in the water distribution systems are piecemeal and mainly related to internal practices of the water utilities (Arregui et al. 2006). Considerable efforts have been made by the IWA Water Loss Task Force (WLTF) to assess the components of the apparent losses and some initial results have been presented by Thornton & Rizzo (2002), among others. Later, Rizzo et al. (2007) proposed an apparent water losses audit based on a pilot zone approach.

The main components of the apparent losses are unauthorised consumption, billing errors and meter under-registration (Rizzo & Cilia 2005; Arregui et al. 2006). According to Criminisi et al. (2009) water metering errors
should be blamed for the biggest part of the apparent losses in a water distribution system. Only meter under-registration is addressed in the present paper because the other causes can be reduced to physiologic levels by good water management administrative practices and controls.

In the supply network, turbine water meters (single or multi jets) are the most common instrument used to measure consumption by users. Water meters are characterised by intrinsic errors that change with the flow rate passed through the meter and can produce under- or over-registration of users’ water consumption.

Generally, the major part of apparent losses due to meter under-registration are related to the percentage of users’ consumption occurring at low and very low flow rates (Criminisi et al. 2009).

The factors affecting the share of water consumption at low flow rates are the low network pressure (a practice usually adopted to minimise real losses in the network) and the presence of users’ tanks (usually fed by a float valve) interposed between the meter and the end user (Thornton & Rizzo 2002; Arregui et al. 2005; Rizzo & Cilia 2005; Cobacho et al. 2008; Criminisi et al. 2009; Fontanazza et al. 2010). This supply scheme is very common in the Mediterranean area to cope with intermittent distribution (Cubillo 2004; 2005). Where water distribution is discontinuous, water resources are often rationed to cope with the scarcity, and the network is subjected to cyclical filling and emptying periods. Users compensate for the intermittent water service by installing private tanks, which collect water during serviceable periods and redistribute it when the public water service is not available. Private tanks greatly affect the hydraulic behaviour of the network, modifying the demand profile of typical domestic users (De Marchis et al. 2010; 2011). The tank is often filled by means of a float valve, which dampens the instantaneous water demand and reduces the water flow rate passing through the meter, increasing metering under-registration. As an alternative to intermittent supply, water utilities can use pressure control to reduce background losses. The control is usually carried out by defining districts in the network where the pressure is controlled by means of pressure reduction valves (PRVs). The PRV works by maintaining a fixed outlet pressure (set point) by automatically adjusting its opening rate. Even if PRVs mainly impact on the reduction of real losses, the drop of network pressure could influence the performance of water meters in the district. Fontanazza et al. (2010) showed that a decrease in network pressure from 2.0 to 0.5 bar can increase apparent losses due to meter under-registration.

In the present paper, a mathematical model for supply systems with private tanks is implemented to identify zones of the network where apparent losses are high and to predict the results of a water meter replacement plan. In accordance with this aim, further developments to a pressure-driven demand simulation model are introduced in the framework of the Global Gradient Algorithm (GGA; Todini 2003; Giustolisi et al. 2008) to properly simulate the use of the tanks. Both PRV and a water metering error model to investigate the influence of pressure control on the apparent losses are considered. The PRV model was newly introduced in the present paper while the water metering error model was transferred from a previous experimental and modelling campaign (Criminisi et al. 2009; Fontanazza et al. 2010).

The proposed mathematical model is applied to a real case study: a district metering area (DMA) supplying 164 domestic end-users with private tanks, the water consumption of which is measured by multi-jet water meters.

**MATERIALS AND METHODS**

The **network simulation model**

The pressure-driven steady-state simulation model of the network, comprising \( n_p \) pipes with unknown flow rates \( Q \), \( n_n \) nodes with unknown water heads \( H \), and \( n_0 \) nodes with known water heads \( H_0 \) is based on the following system of equations:

\[
\begin{bmatrix}
A_{pp} & A_{pn} \\
A_{np} & A_{nn}
\end{bmatrix}
\begin{bmatrix}
Q \\
H
\end{bmatrix}
= 
\begin{bmatrix}
-A_{np}H_0 \\
0
\end{bmatrix}
\]

(1)

where \( Q \) in \( \mathbb{R}^{n_p} \), \( H \) in \( \mathbb{R}^{n_n} \), \( H_0 \) in \( \mathbb{R}^{n_0} \), are column vectors; \( A_{nn} \) in \( \mathbb{R}^{n_n} \) is the diagonal matrix whose elements are the node consumptions evaluated by means of the pressure-driven method described below; \( A_{pn} = A_{np}^T \) and \( A_{np} \) are topological incidence sub-matrices.
with size \([n_p, n_n]\) and \([n_p, n_0]\), respectively, derived from the general topological matrix \(\mathbf{A}_{pp} = [\mathbf{A}_{pp}; \mathbf{A}_{po}]\) of size, \([n_p, n_n + n_0]\), as defined in Todini (2003); \(\mathbf{A}_{pp}\) in \(\mathbb{R}^{\times} [n_p, n_p]\) is the diagonal matrix whose elements are defined as:

\[
\mathbf{A}_{pp}(j, j) = R_i|Q_j|^{n-1} + m|Q_j| \quad \text{for } j = 1, \ldots, n_p
\]  

(2)

where \(R\) is the head loss coefficient which is a function of the pipe roughness, diameter, and length; \(n\) is the exponent which takes into account the flow regime and the head loss relationship chosen (in this paper the Sonnad & Goudar (2007) formulation was used); \(m\) is the coefficient related to minor losses.

The elements of the diagonal matrix \(\mathbf{A}_{nn}\), the water discharge at network nodes \(q = f(P)\), can be defined by means of several formulations (Bhave 1988; Germanopoulos 1985; Wagner et al. 1988; Reddy & Elango 1989; Chandapillai 1991). In this paper that proposed by Criminisi et al. (2009) is chosen and slightly modified as discussed below. This pressure-driven demand model takes into account the user’s private tank, with a float valve that progressively reduces the flow entering the tank during the filling. The model is based on the use of the float valve emitter law to represent the pressure-consumption relationship at a demand node for pressure, \(P\), ranging from the minimum required to have outflow at the node, \(P_{min}\), to the pressure required to satisfy the maximum flow entering the tank, \(q_s\). When the pressure is higher than required, \(P_{min}\), the system outflow is constant and equal to the user demand. Each demand node supplies as many tanks as users considered connected to it. Specifically, the discharge entering the \(k\)-th tank connected to the \(i\)-th node, \(q_{act,i}\), can be obtained as:

\[
q_{act,i}^k = \begin{cases} 
q_{act,i}^k - \tau P_{s,i} \sqrt{2g P_{s,i}} & \text{if } P_{s}^k > P_{s,i} \\
0 & \text{if } P_{min,i} \leq P_{s,i} \leq P_{s,j} \quad k = 1, \ldots, T \\
q_{act,i}^k - \tau h_{max,i} & \text{if } P_{s}^k < P_{min,i} 
\end{cases}
\]  

(3)

where \(c_v\) is the non-dimensional float valve emitter coefficient; \(a_s\), is the valve effective discharge area; \(g\) is the gravity acceleration; and \(T\) is the number of tanks connected to the node.

Although more complex methods were considered in the past to relate the float valve coefficients to its opening rates, depending on the floater position and thus on the water level of the tank (Criminisi et al. 2009), here constant values were used for \(c_v\) and \(a_s\).

Equation (3) must be combined with the tank continuity equation, which can be written for the \(i\)-th node as:

\[
\begin{cases}
q_{act,i} - \sum_{k=1}^{T} q_{act,i}^k = 0 & \text{for } h_{i}^k < h_{max,i} \\
q_{act,i}^k = 0 & \text{for } h_{i}^k \geq h_{max,i}
\end{cases}
\]  

(4)

where \(d\) is the time variable user water consumption; \(V\) is the volume of the \(k\)-th tank having area \(S\) and variable water level \(h\); \(h_{max}\) is the maximum allowed water level in the tank (before the floating valve closes).

Finally, the discharge, \(q_{act}\), and the water consumption of users considered lumped at the \(i\)-th node, \(D_i\), are:

\[
q_{act,i} = \sum_{k=1}^{T} q_{act,i}^k
\]  

(5)

\[
D_i = \sum_{k=1}^{T} d_{i}^k
\]  

(6)

The solution of the system shown in Equation (1), according to Todini (2003), is reduced to the inversion of a symmetrical and sparse matrix.

As underlined by Giustolisi et al. (2008), the head-driven formulation for \(q_{act,i}\), reduces convergence in Newton–Raphson-based algorithms. The lack of convergence and the need to select a good starting point in Equation (1) are typical problems. To overcome the lack of convergence, the following two equations were added:

\[
\mathbf{H}^{i+1} = \lambda^\tau (\mathbf{H}^{i+1} - \mathbf{H}) + \mathbf{H}
\]  

(7)

\[
\mathbf{Q}^{i+1} = \lambda^\tau (\mathbf{Q}^{i+1} - \mathbf{Q}) + \mathbf{Q}
\]  

(8)

where \(\tau\) is the iteration factor, and \(\lambda\) is an under-relaxation factor which accelerates the solution convergence for the non-linear hydraulic problem. The relation between \(\lambda\) and...
convergence is governed by the mean squared error in the mass and energy balance equations while performing the iterative search. When any of these errors increases, $\lambda$ is reduced by a factor set here to 0.7; when both errors decrease, $\lambda$ increases by a factor set here equal to 1.5. The value of $\lambda$ approaches unity as the system converges to the solution. The maximum number of iterations (set equal to 100) is a further control threshold.

To solve the system in Equation (1) the starting point, $H^{p=0}$ and $Q^{e=0}$ was chosen as follows:

$$H^{p=0} = P_{min} + c_0(P_s - P_{min}) + Z$$

(9)

$$Q^{e=0} = \text{pinv}(A_{np})q_{start}$$

(10)

where $P_{min}$ and $P_s$ are vectors whose elements are $P_i$ and $P_{min}$, respectively; $Z$ is the vector of nodal elevation $Z_i$; and $\text{pinv}(A_{np})$ is the Moore Penrose pseudo inverse of $A_{np}$ (Giustolisi et al. 2008); $c_0$ is a constant whose value ranges between 0 and 1.

The PRV model

A model for the PRV, enhanced from that proposed by Prescott & Ulanicki (2003), was developed. A PRV provides the desired outlet pressure (set point) through an hydraulic control loop which is able to settle the valve opening and closing. Therefore for a particular PRV, it is necessary to assess the valve opening as a function of the inlet and outlet pressure values which depend on the network behaviour. The model is based on the following equations:

$$q_3 = \alpha(P_{set} - P_{out})$$

(11)

$$\dot{x}_m = \frac{q_3}{A_{cv}(x_m)}$$

(12)

$$q_m = C_{prv}(x_m)\sqrt{P_{in} - P_{out}} = \frac{1}{m(x_m)}\sqrt{P_{in} - P_{out}}$$

(13)

where $q_3$ is flow entering or leaving the valve control space, the chamber above the main valve element; $\alpha$ is the needle valve speed control setting; $P_{set}$ is the PRV set point; $\dot{x}_m$ is the valve opening velocity; $A_{cv}$ is the cross-sectional area of the control space; $q_m$ is the flow passing through the PRV; $C_{prv}$ is the valve capacity; and $P_{in}$ and $P_{out}$ are the PRV inlet and outlet pressure, respectively. According to Prescott & Ulanicki (2008), it is necessary to specify two relationships: the first (called the characteristic curve) between the valve capacity, $C_{prv}$ and the valve opening, $x_m$, and the second between the cross-sectional area of the control space, $A_{cv}$, and the valve opening, $x_m$. These are provided by the PRV manufacturers or can be experimentally measured.

The network and the PRV simulation models are coupled and tested. At each time step for the proper boundary conditions at the junction nodes and network reservoirs, the network model provides the node pressures and the pipe flows; then $P_{in}$, $P_{out}$ and $q_m$ are determined. For a fixed PRV set-point, $q_3$, $x_m$ and $C_{prv}(x_m)$ are evaluated. The output of PRV model is the PRV minor loss coefficient, $m$, equal to the inverse square of $C_{prv}$ according to Equation (13). Finally, the minor loss coefficient value is fed back to the network model in Equation (2) for the next time step.

The apparent losses module

A specific apparent losses module was developed which was able to quantify the intrinsic meter error related to the meter age, $age$, the flow rate passing through the meter, $q_{act}$, and the pressure at the node, $P$. The complete mathematical formulation of the model and the experimental campaign used to develop it are provided by Criminisi et al. (2009) and Fontanazza et al. (2010). In this section, the essential equations are reported with the related parameters. The estimation of metering error was based on the following equation:

$$q_{mean,i} = f(q_{act,i}) = \begin{cases} 0 & \text{if } q_{act,i} < q_{start,i} \\ q_{act,i} \left[ 1 - \left( \frac{q_{start,i}}{q_{act,i}} \right)^{3} \right] \cos \left( \frac{\pi q_{act,i} - q_{start,i}}{P_{err}} \right) & \text{if } q_{act,i} \geq q_{start,i} \end{cases}$$

(14)

where $q_{mean}$ is the flow measured by the meter installed upstream of the $k$-th tank connected to the $i$-th node [l/h]; $q_{start}$ is the meter starting flow [l/h], defined as the flow that generates motion in the meter when the mechanism is at

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rest; \( \gamma \) is a dimensionless coefficient that takes into account the reduction of metering error with the actual flow rate passing through the meter \( q_{\text{act}} \); \( \text{Per} \) is the semi-period of measurement error oscillation around zero, which accounts for both negative and positive errors depending on passing water flow [l/h].

Namely, Equation (14) is an empirical equation formulated by determining the best fit to the experimental error curves obtained for meters of different ages and for four different test pressures (0.5, 1.0, 1.5, 2.0 bar) (Fontanazza et al. 2010). Those laboratory results revealed that the influence of pressure is negligible on shape parameters \( \gamma \) and \( \text{Per} \) whilst it is relevant to \( q_{\text{start}} \). The relationship between average starting flow and meter age, under four different test pressures, obtained by interpolating experimental results with an exponential law is also given by:

\[
q_{\text{start},i}^k = a_1 e^{a_2 \cdot \text{age}^k} \tag{15}
\]

where \( a_1 \) and \( a_2 \) are two empirical parameters obtained by determining the best fit to the experimental error curves.

Table 1 shows \( a_1 \) and \( a_2 \) values related to the four different test pressures; parameter values for intermediate pressures were obtained by linear interpolation (Fontanazza et al. 2010).

### THE CASE STUDY

The proposed integrated model was applied to a real case study: a small DMA of the distribution network of Palermo, Italy (Figure 1).

The network is about 1.3 km long, and the pipes are made of high density polyethylene, with diameters ranging from 110 to 225 mm. The network presents 40 service connections that supply a total of 164 residential end-users equipped with multi-jet water meters (Table 2). Water meters that are more than 10 years old are in class B, according to ISO 4064 (1995) and Directive 75/33/EEC (1974); others are in class C (ISO 4064 2005).

All the end-users have previously installed private tanks downstream of the revenue meters because the water supply was intermittent in the past. Namely, 34 service connections supply single users with a private tank located on the building roof, while six service connections supply condo collective users with private tanks located under street level.

Since the end of 2009, the DMA was monitored and the input volume and the pressure at the inlet node were measured with a temporal resolution of 30 min. An average

### Table 1 | Parameters of relationship between average starting flow and meter age, under different test pressures

<table>
<thead>
<tr>
<th>Pressure [bar]</th>
<th>( a_1 )</th>
<th>( a_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>4.135</td>
<td>0.045</td>
</tr>
<tr>
<td>1.0</td>
<td>3.717</td>
<td>0.046</td>
</tr>
<tr>
<td>1.5</td>
<td>3.373</td>
<td>0.047</td>
</tr>
<tr>
<td>2.0</td>
<td>3.015</td>
<td>0.050</td>
</tr>
</tbody>
</table>

### Figure 1 | A schematic of the DMA.

### Table 2 | Water meter characteristics

<table>
<thead>
<tr>
<th>Diameter [mm]</th>
<th>1-5 [years]</th>
<th>5-10 [years]</th>
<th>10-15 [years]</th>
<th>15-20 [years]</th>
<th>Total</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>80</td>
<td>31</td>
<td>33</td>
<td>13</td>
<td>157</td>
<td>96</td>
</tr>
<tr>
<td>25</td>
<td>3</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>4</td>
<td>2</td>
</tr>
<tr>
<td>40</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>50</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Total</td>
<td>84</td>
<td>34</td>
<td>33</td>
<td>13</td>
<td>164</td>
<td>100</td>
</tr>
<tr>
<td>%</td>
<td>51</td>
<td>21</td>
<td>20</td>
<td>8</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
water consumption pattern was obtained by recording the water consumption of five single users installing downstream of their private tank a new class C water meter coupled to a data logger.

Water losses due to real leakages in the DMA were evaluated through a noise logger survey and a night flow analysis (WRC 1994; Ulanicki et al. 2000; Hunaidi & Brothers 2007; Tabesh et al. 2009) where the minimum night flow was continuously recorded and analysed. No leaks were found and the background leakage level was assumed to correspond to the minimum measured flow rate due to the residential night water demand. Prior to these measurements, a specific leak detection survey was performed in the users’ plumbing system, and all detected leaks were repaired before the monitoring period started. Water theft did not occur in this district, and meter-reading errors were excluded, as each reading was verified twice by independent operators.

RESULTS ANALYSIS

The hydraulic network model was not calibrated with regard to the pipe roughness because pressure data were available only in the DMA inlet node. The absolute roughness was set to 0.02 mm. The model reliability was proven by applying it to solve a sample network proposed in literature (Fujiwara & Khang 1990). The obtained results showed a good agreement with other models (Savic & Walters 1997).

The node head-discharge model parameters were set to the averages obtained in a field campaign that was carried out in 2007 and 2008 (Criminisi et al. 2009): $c_v$ was set equal to 0.57 and $a_v$ to 2.8 cm$^2$ for each tank. The elevation and the dimension of the tanks were measured in the same monitoring campaign. $P_{T}$ was set 20 m higher than the float valve opening and $P_{min}$ was set equal to the float valve height.

Finally, the PRV model parameters were chosen as follows: the needle valve speed control setting, $\alpha$, was set equal to a constant value, $10^{-6}$; the value of the PRV set point, $P_{set}$, was chosen equal to 17 m to guarantee the tank filling according to the user’s tank position (on the rooftop or underground); and the characteristic curve was selected according to Prescott & Ulanicki (2005):

$$C_{prv} = 451877x_m^4 - 29538x_m^3 + 56696x_m^2 - 0.417x_m$$ (16)

The comprehensive model (hydraulic simulation network, demand and apparent losses models) was applied to the case study and apparent losses at each node were evaluated as the sum of the metering errors of all the connected meters (Figure 2). The apparent loss, i.e. the difference
between the volume measured by the meter and the actual consumed volume, was normalised by the actual consumed volume and expressed as a percentage. In order to summarise the results for each node, Figure 2 shows the median daily losses (the black cross), the 25th and 75th quantiles (the end and the top of the box), representing respectively, the apparent loss that is not exceeded for 25% of the day and the apparent loss that is not exceeded for 75% of the day, the 5th and the 95th quantiles (the ends of the two whiskers) representing respectively, the apparent loss that is not exceeded for 5% of the day and the apparent loss that is not exceeded for 95% of the day. Nodes are ordered against average water meter age and they were divided into three age groups.

Using node 33 as an example, Figure 2 shows that the median apparent loss is 12% and users were supplied for 6 hours (sparse during the day) with high flows that guarantee very low apparent losses (below 2%); only for the 5% of the day (a little bit more than 1 hour), apparent losses are higher than 23%. Looking at node 13, the level of apparent losses is much higher: the median loss is around 38%, apparent losses are lower than 10% only for 1.2 hours and for 6 hours are higher than 55%.

Observing simulation results, for most of the nodes the 25th quantile of metering errors was around $-30\%$ and the 75th quantile is above zero (meaning that the metering error is in favour of the water manager). That condition is due to the high pressures in the network that permit the rapid filling of the private tanks, reducing the metering errors, and to the low flows that enter the tank after the filling, increasing the apparent losses in the rest of the day. The water level in those tanks does not fall much during the day, the valve opens only partially and the flow rate passing through the meter and entering the tank is in the range where the apparent losses are high. Water meters were divided into age groups in order to investigate the impact of ageing on apparent losses: the picture provided is complex and the trend cannot easily be perceived because of many water meters providing higher apparent losses than expected; anyway, looking at the median daily losses, the age class between 15 and 22 years provide much higher apparent losses than the other groups. The age class between 10 and 15 years is the best performing (on average) probably because of a better turbine technology installed in those years.

The same analysis was carried out making the hypothesis that a PRV is installed in the inlet node, reducing the pressure in that node from 43 to 20 m (Figure 3). The reduction of pressure slows down the filling of private tanks for some nodes. This results in several cyclical filling
and emptying processes of the tanks; the related float valves are completely open for a longer time and allow water to pass through the meter at flow rates higher than the starting flow. As a result, the 25% quantile of metering errors is reduced in absolute values. However, with regard to the median metering errors, pressure reduction increases the metering error significantly in some nodes (for instance, nodes 14 and 33) as shown in Figure 3 and in Figure 4 which represents the difference between the average metering errors evaluated for each network node without and with PRV. As demonstrated in Fontanazza et al. (2010) ageing and pressure are both relevant parameters determining meter starting flow. The first is related to starting flow by a non-linear law, with starting flow progressively increasing with the age of the meter. Network pressure also has a linear influence on starting flow, with its effect highest for newer meters and progressively masked by wear and tear during meter ageing: the meter starting flow increases as pressure reduces and this behaviour can greatly affect apparent losses due to metering under-registration.

As Figures 2 and 3 show, the distance between the 25% and the 75% quantiles of the metering errors evaluated for each network node with the presence of a PRV are often higher than those assessed when no PRV is installed, thus confirming that pressure control may have a negative impact on apparent losses. This is not a general rule as the case of node 3 may demonstrate but the analysis shows that the reduction of pressure variation in the network influences the meter error variability: if pressure is maintained in a range where metering errors are high this has a negative impact on apparent losses; in the opposite case, the presence of a PRV may mitigate the apparent losses along with the real ones. The 95th quantile value of metering error is about −30 and −45%, without and with PRV, respectively (Figures 2 and 3). Highest apparent losses are often not related to the same network node thus demonstrating that the change in network pressure distribution may change the identification of most critical node in terms of apparent losses.

The experimental study reported in Fontanazza et al. (2010) showed that the older age meters classes are characterised by a lower relative decrease in starting flow and this behaviour is correctly represented by the model. Even if meter age is considered the most important parameter to carry out a meter substitution plan, older devices can show lower metering errors than newer depending on the hydraulic condition of the network. The correlation between meter age and the amount of apparent losses due to meter under-registration is not simple. The meter may under-register when the tank is usually full and the float valve opens as soon as tank water levels fall. Big consumers (usually condos) are characterised by large tanks and a consumption...
profile that is more distributed through the day; consequently the water level does not fall much, the valve opens only partially and the flow rate passing through the meter and entering the tank is very low leading to high volumes to be apparently lost (Figure 4). Small residential consumers are often characterised by consumption profiles that are concentrated in the evening; for that small period, the tank water level drops, the float valve opens completely and water volumes flow into the tank at a high flow rate thus reducing apparent losses. Even old meters may produce small errors if network pressure and tank filling and emptying work together to increase the flows passing through the meter.

Those results revealed how the complexity of the physical phenomena associated with metering errors in ageing flow meters does not allow meter replacement to be guided by single parameters, such as the meter age or the total metered volume. As discussed in Fontanazza et al. (2002), a composite indicator taking into account three of the most influential parameters that can affect metering accuracy such as the meter age, the total metered volume, and the network pressure could be a useful performance-based tool for prioritising water meter replacement in an urban distribution network.

CONCLUSIONS

This paper shows the results of the application of a mathematical model to assess apparent losses caused by meter under-registration. The model proposed is built up by a hydraulic network model, a PRV model, a pressure-driven demand, and an apparent losses model, and it capable of analysing the complexity of supply systems with private tanks.

With specific regard to the analysed case study, this study highlighted that pressure control, by means of PRVs, may have a negative side effect in the form of an increase in apparent losses. The water meter under-registration increases up to about 50% when a PRV is installed in the inlet node of the DMA. That PRVs may act negatively on apparent losses was demonstrated in the paper but another conclusion was the fact that the stabilisation of pressure may even act positively on metering errors: if pressure is stabilised in a range in which flows passing through the meter are higher, apparent losses are reduced by the presence of the PRV. This evidence is however limited to a few nodes and in general the reduction of network pressures takes the increase of apparent losses as a consequence. Such an effect should be considered when water loss reduction campaigns are designed by the utilities that are increasingly interested in conserving water by reducing pressure and a detailed analysis should be carried out to evaluate the local implications of pressure management. Furthermore, if the flow entering a user’s water system is controlled by a tank, as happens in the analysed case study, the combined influence of pressure, meter age and the tank filling process significantly affects metering errors: these can be negligible if the tank fills and empties cyclically, the water meter is new and the pressure is high; the opposite occurs if the tank does not ever completely empty, the meter is old and the pressure is low.

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