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A genetic algorithm for demand pattern and leakage estimation in a water distribution network

Armando Di Nardo, Michele Di Natale, Corrado Gisonni and Michele Iervolino

ABSTRACT

The sustainable management of water supply networks requires the control of physical pipe leakages, such as those due to junction obsolescence or pipe creeping. These leakages usually increase with the operating pressure, and their discharge is commonly assumed to scale with the power of the pressure. The same functional form is also employed to evaluate leakage occurring in the portion of the network downstream a node. The parameters involved in these relationships may be estimated from field experimental data. However, a sensible fluctuation in their values is observed, and therefore the definition of a suitable leakage law represents a major source of uncertainty in water network modeling. In the present paper, the estimation of the leakage law parameters is carried out simultaneously to the hourly demand pattern. To this aim, a hydraulic network model coupled to a genetic algorithm is employed to minimize the deviation between predicted and measured time series of pressure and flow at a small number of sites of the network. A field test case is analyzed to show the effectiveness of the proposed procedure.

Key words | demand models, genetic algorithms, leakages, optimization, water distribution networks

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NOTATION

c	parameter of leakage law [$m^{3-\gamma}/s$]
C_d	per-capita hourly water demand [m^3/s/inhab.]
$\mathbf{d}$	data vector [dimensions may vary]
D	node outflow [m^3/s]
$G(\cdot)$	symbolic operator of a generic mathematical model [dimensions may vary]
L	leak discharge [m^3/s]
m	number of parameters [-]
$\mathbf{M}$	parameter vector for G model [dimensions may vary]
N_1	exponent of leakage law [-]
n_h	number of hours in the time pattern [-]
n_{mP}	total number of pressure stations [-]
n_{mQ}	total number of flow stations [-]
$n_{u,i}$	number of users supplied by i-th node [-]
OF	objective function [-]
P	node pressure head [m]
$\mathcal{P}_{\text{cross}}$	crossover probability [-]
Q	volumetric flow [m^3/s]
γ	exponent of leakage law [-]

SUPERSCRIPT

k hour index

SUBSCRIPTS

1	pertaining to network zone 1
2	pertaining to network zone 2
C	computed
i	node index
M	measured

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INTRODUCTION

Leakages in water distribution networks represent a relevant technical and management problem for industrialized countries. Leakage levels above 50% of the supplied volume (OECD 2002) are nowadays unsustainable from both an environmental and economic standpoint. Some countries have been facing pipe leakages for many years, promoting research, special laws and the spreading of best practices (UKWIR 1999); many others, however, addressed the problem with enormous delay (Di Nardo *et al.* 2013a). The worldwide relevance of the topic (UNESCO 1999) prompted many important associations (AWWA 1999; Lambert & Hirner 2000) to promote best operating technologies among water utilities.

Water losses are commonly split into two components (Alegre *et al.* 2000): physical or real losses due to pipe or network component bursts, and apparent losses mainly due to unauthorized consumption or metering inaccuracies. The former are addressed in this paper.

The discharge L of a pointwise leak is commonly expressed as a power function of the pressure head P (Goodwin 1980; Sendil & Al-Dhowalia 1992; Lambert 2001):

$$L = cP^{\gamma} \tag{1}$$

through the dimensional parameter c and the exponent γ. From the literature, under different field or laboratory conditions, a strong variability in the value of γ is observed (Lalonde 2005; Levin 2005; Sturm & Thornton 2005; Covas *et al.* 2006). May (1994) suggests to compute the leakage as the average of two orifice flows, one from an ideal one ($\gamma = 0.5$) and the other from an orifice whose area linearly increases with the pressure head ($\gamma = 1.5$). The above model is also consistent with the observations of Khadam *et al.* (1991), which measured $\gamma = 1 \div 1.43$. Lambert (2001), accounting for the dependence of flow velocity on the Reynolds number and assuming a linear relation among orifice area and pressure, postulates that γ may reach values as high as 2.5, as observed in the experimental results of Hiki (1981) and Lambert *et al.* (1999). Further experimental studies have shown that γ can vary between 0.5 and 2.79 (Farley & Trow 2003) or between 0.42 and 2.30 (Greyvenstein & van Zyl 2006), these differences being mainly related to burst type (round hole, longitudinal crack, circumferential crack, corrosion cluster, etc.) and to the pipe material (plastic, steel, concrete, etc.).

A functional form similar to Equation (1) is also employed to scale the leakage occurring in a district of the network located downstream from a demand node:

$$\frac{L_1}{L_2} = \left(\frac{P_1}{P_2}\right)^{N_1} \tag{2}$$

in which L_1 and L_2 denote the leakage discharges corresponding to the average pressure heads P_1 and P_2, respectively. Based on field data from the UK, Japan, Australia, Brazil, Canada, Malaysia, New Zealand and the USA (Thornton *et al.* 2005), the N_1 exponent was found to vary between 0.5 and 1.5, but it may occasionally reach values as high as 2.5 (Thornton & Lambert 2005).

From Equations (1) and (2), it is seen that pressure management, avoiding un-needed pressure excesses, may reduce pipe leakage during the day, with the twofold benefit of leakage (Germanopoulus & Jowitt 1989; Jowitt & Xu 1990; McKenzie & Wegelin 2005) and burst frequency reduction (Pearson *et al.* 2005). The estimation of the parameters appearing in Equations (1) and (2) is therefore mandatory for any leakage control and management policy (Almandoz *et al.* 2005). The evaluation of the above parameters based on an experimental campaign may be very expensive and time-consuming, and it is obviously impossible in the design of new networks. On the other hand, the variability of the parameter values found in the literature does not permit any reliable a-priori estimate.

A review of leakage estimation models by means of different mathematical approaches, based on continuous flow and/or pressure measurements, is reported by Buchberger & Nadimpalli (2004). The authors also propose a novel method based on statistical analysis of flow readings.

An alternative methodology is proposed in this paper for the estimation of c and γ, which relies on the solution of an inverse modeling problem where demand and leakage pattern are assumed as unknowns. This represents an alternative to previous leakage estimation methods relying on the preliminary knowledge of demand pattern (Almandoz *et al.* 2005), which is, however, not always available. The inverse problem is solved by means of a genetic algorithm (GA), in which fitness function scales as the reciprocal of the deviation between predicted and measured flow and pressure time series in a few points of the network. The estimation procedure was developed and tested using field measures from the Monterusciello water network (in a suburban area of Pozzuoli in Italy).

MATERIALS AND METHODS

M is the vector of m parameters of the generic mathematical model G, and **d** is the vector of output data resulting from the forward operator:

$$G(\mathbf{M}) = \mathbf{d} \tag{3}$$

the inverse problem consists in computing **M**, once the values of **d** are given, or symbolically:

$$\mathbf{M} = G^{-1}(\mathbf{d}). \tag{4}$$

In the present case, the forward operator is represented by the set of the quasi-steady equation of momentum and mass conservation, written for each network pipe and junction, respectively, in which all the geometric and hydraulic properties are considered known.

To express the outflow at each node, both user demands and leakages are accounted for. The former are assumed to be pressure-independent and the latter are expressed through a power-law of node pressure head, according to Equation (1). Assuming an hourly-based variation of water demand, and introducing the following hypotheses: (a) water consumption by the users independent from the pressure; (b) homogeneous distribution of per-capita daily freshwater request; and (c) homogeneous values of the parameters in the pressure-leakage law, the discharge D_i^k of i-th node on the k-th hour can be evaluated as:

$$D_i^k = n_{u,i} c_d^k + c\left(P_i^k\right)^{\gamma} \tag{5}$$

where $n_{u,i}$ is the number of users connected to the i-th node and c_d^k is the k-th hourly per-capita water demand, whose daily average represents the daily per-capita freshwater request, and P_i^k represents the pressure head at node i in the k-th hour. The validity of the combined demand-leakage model, Equation (5), has been widely recognized to be reasonably accurate for water supply network modeling (e.g. Wu *et al.* 2009, 2010). It is worthy of note that the present form of Equation (5) considers a constant value of the c coefficient for all the network nodes. This assumption appears to be appropriate for relatively small networks as those considered in the following case-study. In principle, the c coefficient may depend on the number of users and/or the total length of the pipes supplied by each node; this effect could be significant for larger networks. This aspect will be the object of future research developments.

Denoting with n_h as the number of time-steps, Equation (5) contains $(n_h + 2)$ parameters, namely the c_d^k coefficients and the two parameters of Equation (1), c and γ.

In the present application, the data set **d** is represented by a measured time series of hourly-averaged flow, $Q_{M,i}^k$, and pressure heads, $P_{M,i}^k$, at the i-th network station at the k-th hour; in total, n_{mQ} and n_{mP} flow and pressure stations are considered, respectively.

For any given set of parameter values, once the hydraulic simulation of the network has been performed, the following objective function (OF) can be evaluated:

$$\mathrm{OF} = \frac{\sqrt{\sum_{k=1}^{n_h}\left[\sum_{i=1}^{n_{mQ}}\left(\frac{Q_{C,i}^k - Q_{M,i}^k}{Q_{M,i}^k}\right)^2 + \sum_{i=1}^{n_{mP}}\left(\frac{P_{C,i}^k - P_{M,i}^k}{P_{M,i}^k}\right)^2\right]}}{\sqrt{n_h}\left(n_{mQ} + n_{mP}\right)} \tag{6}$$

in which $Q_{C,i}^k$ and $P_{C,i}^k$ denote the computed flow and pressure head at node i for the k-th hour, respectively.

Since OF represents a measure of the deviation of computed values (subscript C) with respect to the observed ones (subscript M), the optimal choice of the above parameters set is naturally defined as the one minimizing the OF. The above minimum is sought for by means of a GA, implemented with MATLAB© (by The MathWorks). As far as network simulations are concerned, a hydraulic model capable of handling both fixed and pressure-dependent nodal outflow is needed for the solution of the problem, due to the dependence of the second term in Equation (5) on the pressure head. In the present analysis, the EPANET code (Rossman 2000), has been chosen since it has already been applied to the analysis of networks with pressure-dependent leakages (see among many others Alonso *et al.* (2000)).

GAs (Goldberg 1989) have been recognized as a powerful and flexible tool for optimization problems, and they have been diffusely used for problems like network calibration, design, rehabilitation, valve insertion, and instrument positioning. There were various reasons for choosing to use a GA: (a) the solution space was very large and complex with $(n_h + 2)$ optimization variables; (b) the OF was strongly non-linear and constrained; and (c) no mathematical analysis of OF was available. As known, in these cases, the optimization problem is too computationally intensive to rely on traditional optimization algorithms, whereas, evolutionary algorithms are suitable to find a near-optimal solution (Savic & Walters 1997). Moreover, a GA is implicitly parallel (Holland 1992), because the algorithm investigates the solution space starting from different initial points for each generation, and this feature is very useful because the OF Equation (6) may be affected by multiple local minima.

Among many others, recent contributions to the topic can be found in the literature (Wu & Simpson 2002; Kapelan *et al.* 2003; Pezzinga & Pititto 2005; Di Nardo *et al.* 2013b). GA move from the definition of an appropriate coding for a candidate solution to the problem, by which each solution can be converted into a 'genetic code'. An initial group of candidate solutions is generated at the beginning of the procedure and repeatedly selected according to the fitness of each solution (Rudolph 1994), up to the fulfillment of a suitable termination criterion, corresponding to a reasonably small value of the OF. In the present application, the fitness has been chosen to be proportional to the reciprocal of the OF. It is worth mentioning that the standard formulation of a GA has been enhanced in the present application with linear fitness scaling in order to avoid early domination of extraordinary strings and to encourage a healthy competition among equals, and with elitism ensure that the best individuals are never lost when moving from one generation to another.

Single-precision real coding is used in the present application, so that each individual of the population is constituted by a sequence of n_h genes (the c_d^k per-capita demands), and two more genes representing the c and γ parameters of the leakage law Equation (5).

Based on preliminary sensitivity analysis, population size was taken as 100: among them, 80 individuals are subjected to scattered crossover with a crossover probability $\mathcal{P}_{\text{cross}} = 0.8$, while two elitist individuals are preserved from both mutation and crossover.

The following termination criterion was employed: no change in the OF value after 80 generations, nor for 80 more generations after replacement of non-elitist individuals with random ones. Such a criterion has shown to considerably reduce the risk of slow convergence due to progressive reduction in population diversity during the evolution. A flowchart of the proposed algorithm is represented in Figure 1.

Case study

The proposed methodology was applied to the water supply network of Monterusciello (Figure 2), a district of Pozzuoli in the Province of Naples (Italy), which comprises about 4,500 properties, distributed in 267 buildings, with about 16,000 habitants (Di Nardo *et al.* 2013a). The considered district contains about 4,700 habitants. Apart from a few public buildings, the area exhibits a prominent residential character. The network was built in a few years during 1980s with steel pipes, without any further modification of the initial layout.

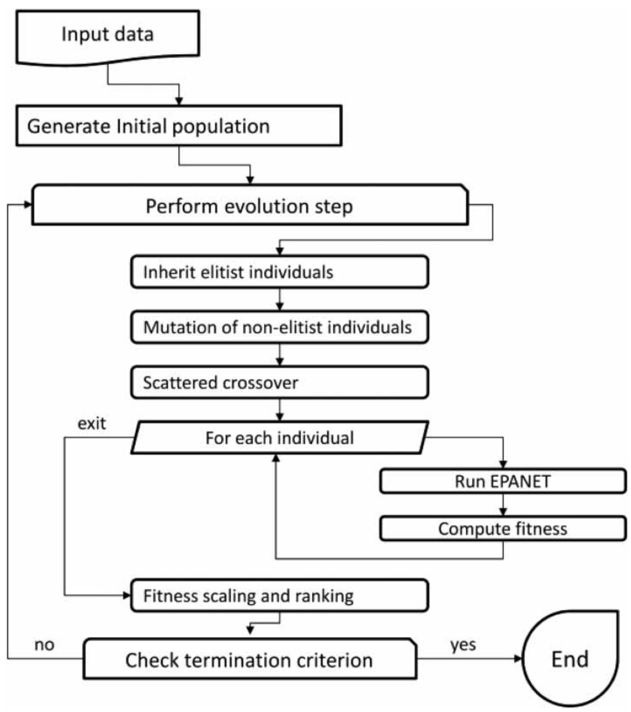

Figure 1 | Flowchart of the algorithm.

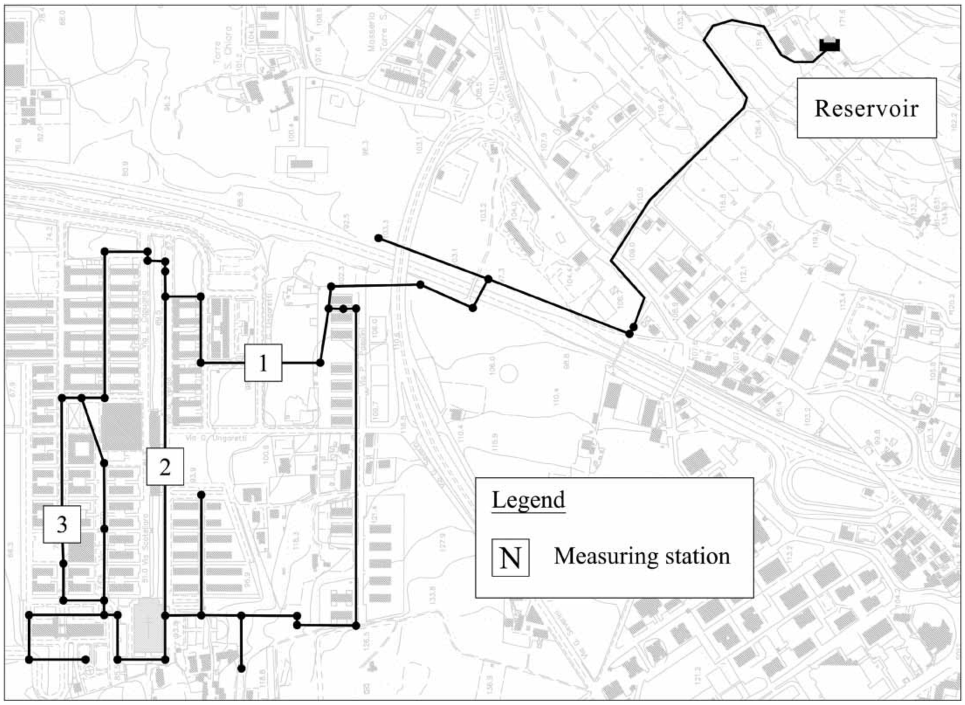

Figure 2 | Monterusciello water network model.

The network was equipped with discharge and pressure measuring instruments under the research project HYDRA-NET (funded by INTERREG IIIB-MEDOCC program, as reported in Di Nardo *et al.* (2013a)). Some of the features of this site make it a suitable candidate for the application of the proposed methodology, while it may adequately represent a typical network serving a small residential suburb. The proposed estimation procedure is, indeed, thought to be potentially applicable to any water supply network of similar size and features.

In Figure 2, the position of the existing measuring stations, which allow flow and pressure continuous monitoring, is also depicted. These prototypal devices, known as FCS (fluid control system), are connected to an advanced remote-control system for data logging and storage (Di Nardo *et al.* 2013a).

RESULTS AND DISCUSSION

The methodology proposed has been validated on simulated data and then tested on field data, as reported in the following section.

Preliminary analysis and validation

The following sources of uncertainty may affect the performance of the outlined procedure: inadequacies of the assumptions in the demand and leakage models; insufficient number and/or wrong position of measuring stations; limited measuring accuracy; random and/or systematic measuring errors; and unrealistic choice of the remaining model parameters (e.g. roughness).

In any field application, all of the above elements will affect the overall performance of the estimation procedure. In order to assess the performance of the parameter estimation independently of the above source of uncertainties, the proposed algorithm was preliminary applied to a synthetic data set.

To perform this benchmark, hourly per-capita water demand c_d^k and leakage parameters c and γ were arbitrarily given the values reported in the first row of Table 1. The subsequent simulation of the network allowed researchers to obtain a synthetic data set, made by the time series of simulated flow and pressure at the locations of the existing measuring stations. The estimation procedure was then carried out with these synthetic data as measures, until the fulfillment of the termination criterion. The comparison between the estimated

Table 1 | True vs. estimated parameters

c_d^k [cm^3/s/inhab.]	1	2	3	4	5	6	7	8	9	10	11	12
True	0.866	0.943	0.000	0.000	0.838	0.820	0.825	0.861	4.452	5.024	4.750	4.303
No noise	0.863	0.941	0.000	0.000	0.833	0.819	0.820	0.855	4.434	5.000	4.731	4.289
Random noise	0.982	1.068	0.134	0.134	1.024	1.000	0.939	1.001	4.489	5.226	5.130	4.428
Constant noise	0.931	1.006	0.148	0.147	0.903	0.889	0.893	0.921	4.398	5.002	4.710	4.239

c_d^k [cm^3/s/inhab.]	13	14	15	16	17	18	19	20	21	22	23	24
True	2.615	1.679	1.718	3.967	1.481	0.815	0.846	1.870	1.618	0.879	1.398	1.987
No noise	2.603	1.677	1.712	3.963	1.475	0.812	0.840	1.864	1.612	0.878	1.395	1.979
Random noise	2.608	1.716	1.781	3.949	1.598	1.001	0.958	1.916	1.763	0.996	1.481	2.155
Constant noise	2.558	1.677	1.714	3.895	1.489	0.882	0.911	1.850	1.620	0.944	1.422	1.954

	c	γ
True	0.012	0.700
No noise	0.019	0.599
Random noise	0.004	0.888
Constant noise	0.003	0.931

values of the parameters and the corresponding 'true' ones revealed that both the temporal variation of the hourly demand coefficients (Figure 3) and the leakage law (Figure 4) are estimated with very good accuracy.

Figures 3 and 4 prove the accurate evaluation of the parameters involved in the demand/leakage model; as a consequence, the total leakage discharge of the network is evaluated with a satisfactory degree of accuracy (Figure 5). It is worth remarking that the accurate estimation of these quantities would represent the main concern from the water utility standpoint.

Moreover, the sensitivity of the results respect to the random initial population composition was verified and found to be negligible. The above results demonstrate the effectiveness of the proposed algorithm, and indirectly suggest that number and location of the measuring station is adequate to the scope. Indeed, an insufficient number of measuring stations and/or their wrong positioning (e.g.

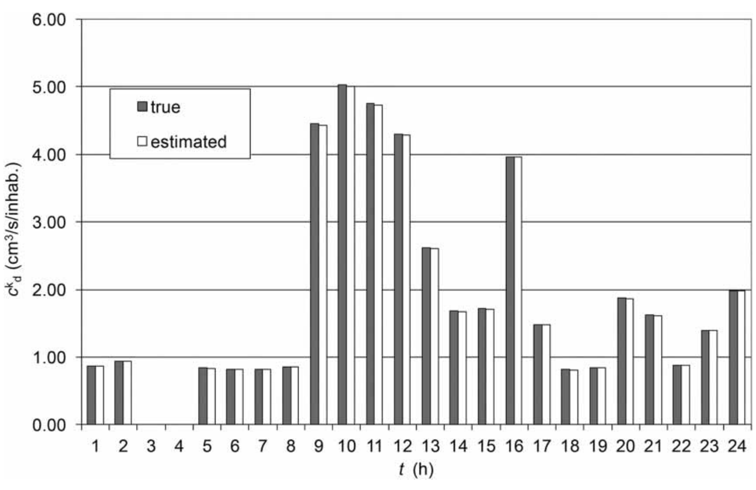

Figure 3 | True vs. estimated hourly per-capita water demands (no noise).

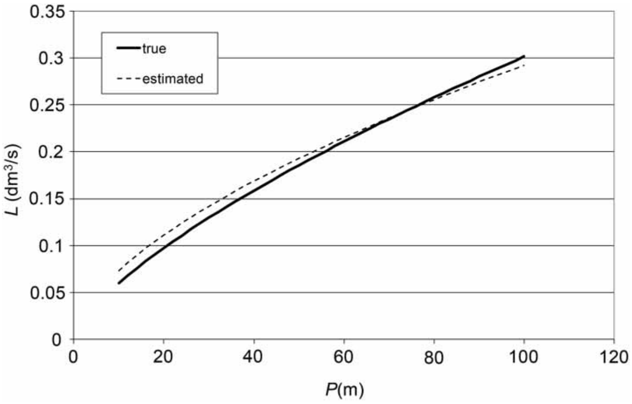

Figure 4 | True vs. estimated leakage law (no noise).

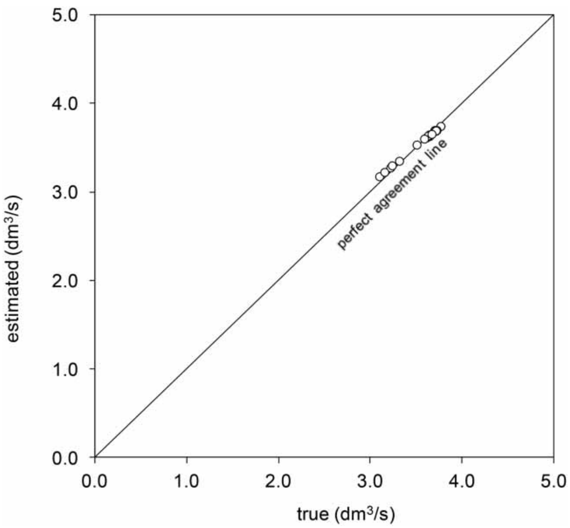

Figure 5 | True vs. estimated hourly wasted discharge (no noise).

distinct stations with highly correlated signals) implies a reduction of the available information about the network status (Zhang & Huang 2009). In principle, also for the proposed methodology, the effectiveness of the GA selection procedure may be significantly affected by inadequate information caused by either correlated measures or improper location of measuring stations.

A further analysis has been carried out to assess the sensitivity of the proposed estimation procedure with respect to the presence of instrumental noise in the measurements. This analysis may be thus interpreted as a measure of the robustness of the procedure. It has to be remarked that the modeling errors are still not included in the analysis at this stage. Again, a synthetic data set was generated by perturbing the previous one with a suitable noise function. Two noise functions have been taken into account, namely: (a) a white noise with root-mean-square (RMS) equal to 10% of the measured value; and (b) a constant noise of −10% of the 'true' value, representative of random or constant errors of the measuring devices, respectively. The estimation was again repeated until the fulfillment of the above termination criterion, which was achieved after a considerably higher number of generations.

True and estimated values of the parameters are compared in the following figures for the case of white noise perturbation. Again, the temporal behavior of the discharge coefficients is well represented in the estimated pattern (Figure 6), even if the agreement is worse than in the previous case. Relative errors with respect to the true value ranges from 0.5 to 23%, with an average value of 11%, which is close to the noise RMS. Despite the value of the leakage law exponent, which is greater than the true one, total leakage discharge of the network is systematically underpredicted, as can be seen by Figures 7 and 8.

Figures 9 and 10 compare true and estimated values of the parameters in the presence of systematic perturbation of the measured data. Again, the time series of the discharge coefficients are well reproduced despite the noisy measurements (Figure 9), with errors ranging from −2.2 to 8% with an average of about 5%, i.e., smaller than the perturbation amplitude. Similarly to the previous case, the estimated leakage law exponent is greater than the true one, but a general underestimation of total leakage discharge of the network is observed (Figures 10 and 11). The detail of the true and estimated values of the parameters in all the previous conditions is reported in Table 1.

Based on these results, we can conclude that the estimation of the discharge coefficients can be achieved with an accuracy comparable to the measured one. On the other hand, the estimation of the parameters of the leakage law appears to be rather sensitive to the presence of noise in the experimental data. In view of the application of the estimation procedure to real field data, any effective treatment to reduce the data noise should be therefore encouraged.

Application to field data

The application of the proposed parameter estimation to real-site measures has to deal with the sources of

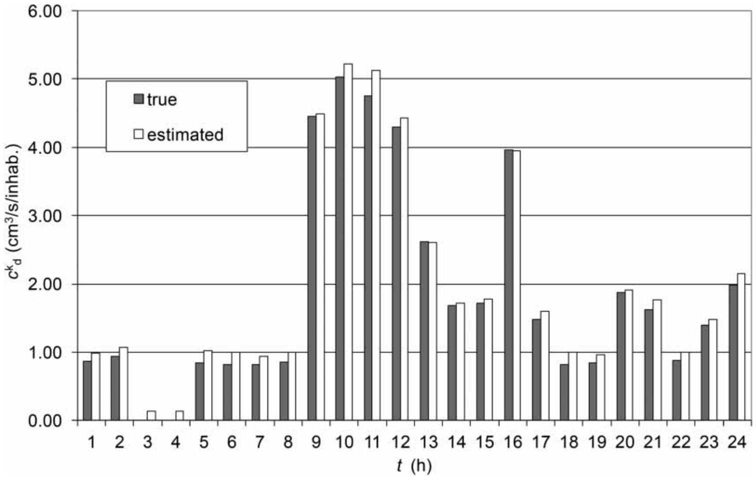

Figure 6 | True vs. estimated hourly per-capita water demands (random noise).

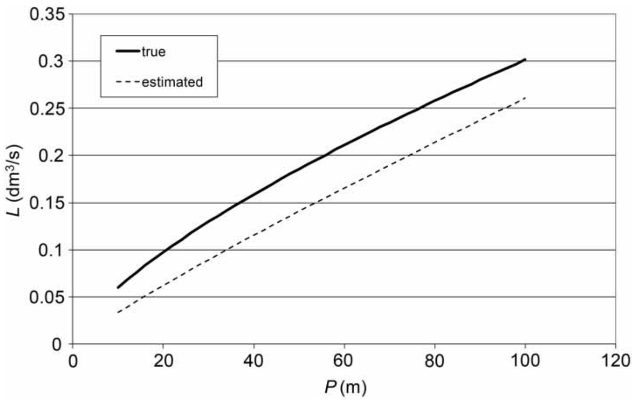

Figure 7 | True vs. estimated leakage law (random noise).

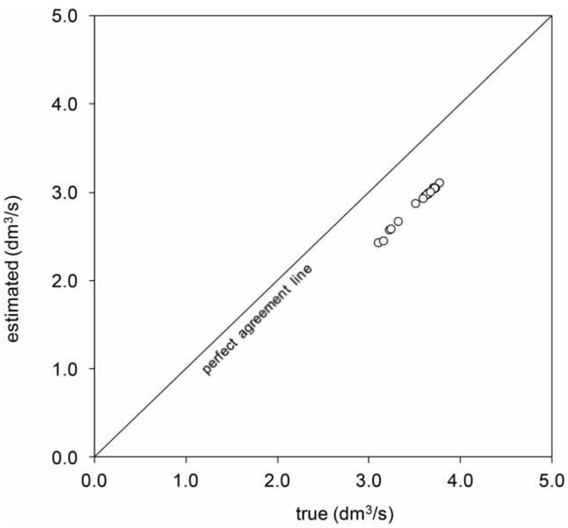

Figure 8 | True vs. estimated hourly wasted discharge (random noise).

uncertainties listed in the previous paragraph. To this aim, we consider that:

- the spatial homogeneity of water usage in the considered water supply network suggests that the assumptions in the demand and leakage models are adequately fulfilled, especially if average behavior is concerned;
- the results of the preliminary analyses suggest that the number and the position of measuring stations is adequate and allows correct estimation of the parameters, provided that noise effects in the measures are limited; and
- based on the detailed knowledge of the infrastructure by the local water utility, the mathematical model may be considered to represent an effective reproduction of the real network.

In order to define the data set needed for procedure application, the time series of pressure and flow at the three measuring locations were acquired for a whole month by means of the installed FCS devices.

Daily measures were processed with an hour-locked average, with the twofold aim: to characterize the average behavior of the users served by the present network (neglecting daily fluctuations) and to reduce the effect of noise in the measures. Since preliminary data analysis showed that working days and holidays were characterized by systematically different behaviors, only the working days were then extracted from the original data set, and averaged in order to define the 'average working day'.

The estimation procedure was then carried out using the average flow and pressure time series as measured data. The

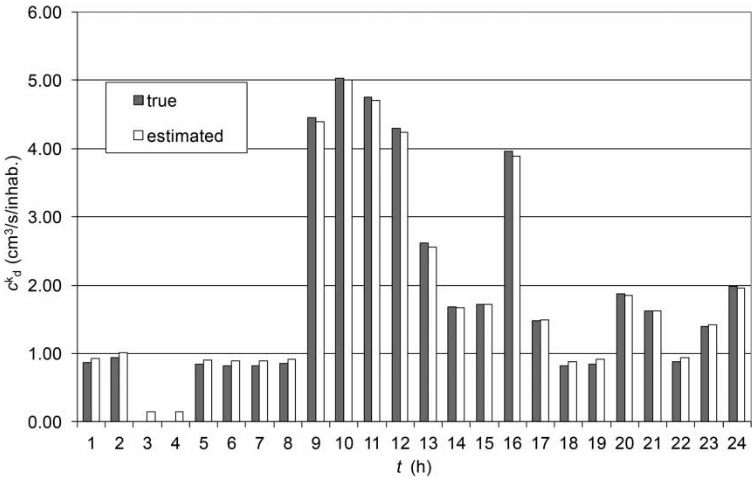

Figure 9 | True vs. estimated hourly per-capita water demands (constant noise).

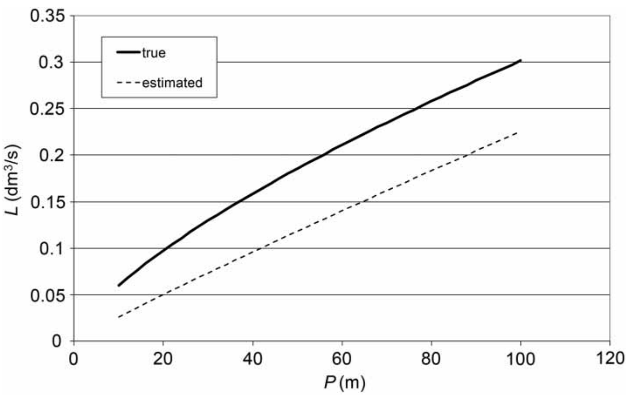

Figure 10 | True vs. estimated leakage law (constant noise).

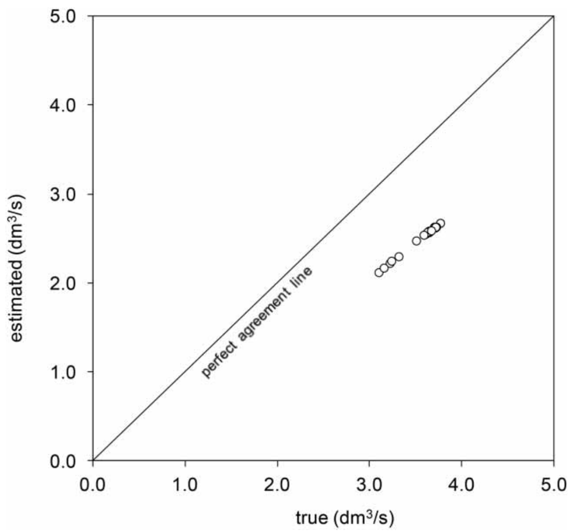

Figure 11 | True vs. estimated hourly wasted discharge (constant noise).

termination criterion was satisfied with a value of the OF equal to 0.065.

Figures 12 and 13 compare measured and computed flow and pressure time series, respectively. As can be observed, the time behavior of the flow is very well reproduced for both the two measuring stations on the feeding pipes (Stations 1 and 2) and the one on the internal pipe (Station 3). The three peaks observed in the measured inflow are clearly recognizable in the corresponding simulated time series, and the repartition of the inflow between the two feeding pipes is also well reproduced.

In addition, the temporal behavior of the pressure head (Figure 13) exhibits a reasonable qualitative agreement, despite some systematic quantitative differences existing, especially for the measuring Station 3. The computed values of the demand and leakage parameters are listed in Table 2: indeed, the estimated per-capita net freshwater daily consumption (about 186 l) and hour peak coefficient (about 1.6) agree with the values reported in the literature for residential areas with comparable population size (Stephenson 1998). The exponent in the leakage law, Equation (1), being relatively close to the value $\gamma = 0.5$ of an ideal orifice flow, falls within the range reported in the literature.

Based on present results, the Monterusciello network seems to be characterized by a rather constant value of the leakage intensity, with small variations from one hour to another (1.8–1.9 l/s), as reported in Figure 14. Leakage amounts to about 18% of the inflow volume, a value consistent with the

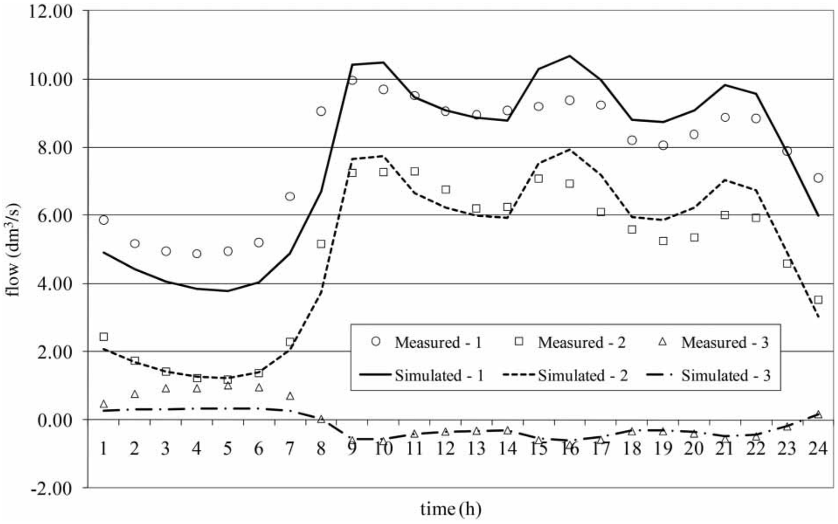

Figure 12 | Measured and computed flow time series.

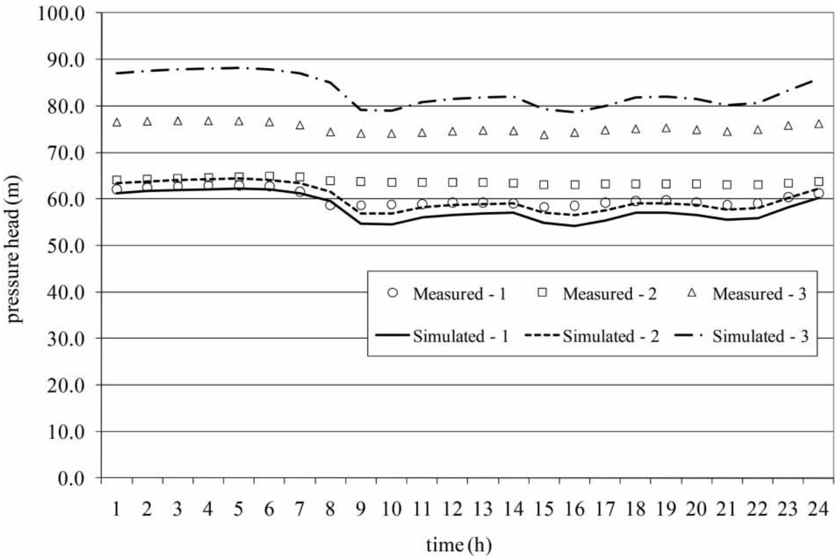

Figure 13 | Measured and computed pressure time series.

Table 2 | Estimated values of model parameters

c_d^k [cm^3/s/inhab.]	1	2	3	4	5	6	7	8	9	10	11	12
	1.01	0.83	0.70	0.63	0.61	0.70	1.00	1.71	3.27	3.30	2.87	2.71
c_d^k [cm^3/s/inhab.]	13	14	15	16	17	18	19	20	21	22	23	24
	2.61	2.58	3.22	3.37	3.08	2.59	2.56	2.71	3.02	2.91	2.18	1.43
c	0.011											
γ	0.569											

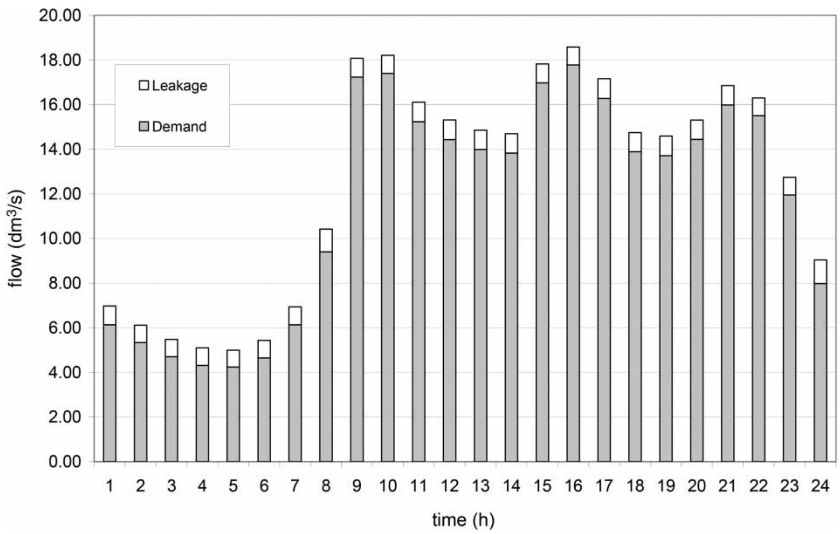

Figure 14 | Computed demand and leakage discharges.

relatively young age of the hydraulic network. Future research efforts will be devoted to evaluating the effect of pressure regulation as a tool for reducing the present leakage level.

CONCLUSION

A procedure for the simultaneous estimation of leakage and demand pattern in a metered water supply network has been presented. The procedure is based on a simplified model of water demand and leakage, in which the first is assumed pressure-independent and the second is expressed through a power-law. Based on these assumptions, the unknown model parameters are assumed as the unknowns of an optimization problem, tending to minimize the deviation from observed time series of flow and pressure at some network locations. The minimum has been sought by coupling a hydraulic network model with an enhanced GA.

When applied to a synthetic data set, the proposed estimation correctly individuates the true values of the demand parameters and of the leakages. When synthetic data are perturbed with random or constant noise, the accuracy in the estimation of demand parameters has been shown to be comparable to the noise amplitude, despite the observed systematic underestimation of the leakages.

Based on these preliminary results, the proposed procedure has been applied to field data coming from a portion of the network of the residential suburb of Monterusciello, in Italy, where a pilot site has been recently realized. A suitable hour-locked averaging procedure has been applied to daily data in order to reduce the effect of noise. The results of the estimation appear to be consistent with the features of the network, encouraging the future application of the proposed procedure to other case studies of similar size and operating conditions. Future research intends to investigate the effects of the number of users supplied by each node on the parameters of Equation (1).

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