

Optimal Location and Setting of PRVs in WDS for Leakage Minimization

Carmine Covelli¹ · Luca Cozzolino² · Luigi Cimorelli¹ ·
Renata Della Morte² · Domenico Pianese¹

Received: 31 October 2014 / Accepted: 26 January 2016 /
Published online: 15 February 2016
© Springer Science+Business Media Dordrecht 2016

Abstract Water loss is an issue that affect Water Distribution Systems (WDSs) very often, especially when aged and high pressure occurs. Pressure reduction valves (PRVs) can be used as devices to reduce as much as possible the water losses within the network. Indeed, for a given number of PRVs, the daily volume of water lost from the network can be reduced minimizing the pressure through a proper choice of valve positions as well as their settings. In this paper, a methodology for the optimal number, positioning and setting of PRVs is presented. In the proposed methodology, a genetic algorithm is coupled with a physical modelling of leakage from joints and a simplified and yet realistic hydraulic simulation of the WDS. The proposed methodology is demonstrated using two WDSs examples. Comparisons with a more extreme and complicated hydraulic modelling, already proposed by authors in previous work, are also performed in the first case study in order to validate the proposed methodology. These comparisons demonstrate that the methodology proposed in this work performs fairly well when compared to similar approach that uses a more sophisticated hydraulic model. As a consequence, it revealed to be a good tool for the optimal positioning and sizing of PRVs within WDS aimed at reducing the background leakages even when the WDS is characterized by complex geometry and topology.

Keywords Optimisation · Valves location · Valves setting · Pressure reduction valves · Leakage · Water distribution systems

1 Introduction

Water losses within a Water Distribution System (*WDS*) may occur at pipeline joints (background leakages), and/or at holes and longitudinal or circumferential cracks (Tucciarelli et al.

✉ Carmine Covelli
carcovel@unina.it

¹ Department of Civil, Architectural and Environmental Engrg., Federico II University, Naples, Italy

² Department of Engrg., Parthenope University, Naples, Italy

1999; van Zyl and Clayton 2005; Nicolini and Zovatto 2009; Puust et al. 2010). In particular, when pipelines are connected by joints, a considerable part of the water losses from the network occur because of the incorrect assembly of joints or the fatigue and ageing of the material used to ensure a watertight seal (Covelli et al. 2015).

In all cases, the leakage discharges depend on the local pressure and can be modelled by means of appropriate pressure-discharge equations. This observation suggests that the correct management of undetectable leaks could be achieved by minimising the nodal pressures throughout the *WDS* (Report 26 1985; Thornton 2003; Marunga et al. 2006; Walski et al. 2006; Ulanicki et al. 2008; Hunaidi 2010; Lambert and Thornton 2011; Xu et al. 2014). This purpose can be accomplished by means of flow or pressure control devices (*Throttle Control Valves (TCVs)* or *Pressure Reduction Valves (PRVs)* or by using small- or micro-turbines or *Pumps as Turbines (PaT)* (Ramos et al. 2010)), able to totally replace the *PRVs* or to be located in parallel or in series with them. In particular, *PRVs* should be placed and set to approximate an ‘*ideal condition*’ in which the nodal pressure heads are very close to the minimal values strictly required to satisfy the local water demand. Because of the problems introduced by the transients during valve operation (Prescott and Ulanicki 2008; Abdelmeguid 2011; Abdelmeguid et al. 2011), very often fixed set-point *PRVs* (i.e., standard *PRVs* that are able to regulate a high varying pressure into a lower and stable downstream pressure, regardless of changes of users demand) are preferred.

Obviously, the optimal positioning and setting of the valves are strongly connected tasks. In the literature, many works have been devoted to the optimisation of valve setting for fixed valve positions, considering stationary water demands (Germanopoulos and Jowitt 1989) or time variable discharges required at the nodes (namely, *EPS = Extended Period Simulation*, Jowitt and Xu 1990; Germanopoulos 1995; Vairavamoorthy and Lumbers 1998; Tabesh and Hoomehr 2007; Ulanicki et al. 2000; Liberatore and Sechi 2009; Dai and Li 2016). In contrast to the above mentioned researchers, other authors have considered the problem of the optimal valves positioning (Savić and Walters 1996; Reis et al. 1997; Araujo et al. 2006).

In order to minimise simultaneously the number of valves and the volume of water lost during a daily cycle (crudely represented by means of three different steady hydraulic conditions), Nicolini and Zovatto (2009) used a multi-objective *Genetic Algorithm (GA)* (NSGA-II), and considered time variable setting values. However, their approach does not appear suitable to identify the number of valves to be used. In alternative, it is possible to consider an approach based on the minimisation of the total costs connected to the water leakage and the pressure management. In fact, we observe that by increasing the number of valves there is a reduction of the volume of water dispersed within the *WDS* and the costs associated with these losses, but there is also an increase of the costs of installation and maintenance of valves. As a result, it is possible to identify the number and the setting of valves that are able to allow a reduction of the whole costs related to both the volumes of water dispersed and valves.

Starting from these observations, a novel approach is proposed in this work aimed at identifying the number, positions and settings of fixed set-point *PRVs* for the reduction of the daily and yearly water losses from joints, and correspondingly costs due to the loss of the profits that could be earned from the sale of the water lost through background leakage. The approach is based on the coupling of a *GA* and a simplified hydraulic model for the analysis of *WDS* that at each time step of the *Extended Period Simulation (EPS)* analysis, employs a ‘*feedback mechanism*’ in order to account for the water lost from the pipeline joints modelled with a pressure-discharge relationship. This ‘*feedback*

mechanism' consists on an iterative procedure that allows accounting for the presence of losses from pipeline joints, without actually modelling them as emitters. Indeed, the amount of water lost from a joint depends upon the value of the local pressure, and then, for a realistic modelling of the network, the pipeline joints should be modelled as emitters nodes (Covelli et al. 2016). However, this extreme network modelling would lead to time consuming hydraulic simulations. *Vice-versa*, the feedback mechanism proposed in this work allows reducing the number of nodes to be considered in the network model, leading to a simplified but efficient *WDS* simulation.

In order to test the proposed methodology, the *GA* coupled with the simplified hydraulic simulator proposed in this paper is compared to the *GA* coupled with more sophisticated hydraulic modelling considered in previous work by Covelli et al. (2016). This comparison is carried out using the same study case already considered in the above cited paper, consisting in a small but realistic *WDS*. Then the proposed methodology is applied to a second case study taken from literature in order to show the capability of the proposed methodology in the reduction of water losses.

2 Proposed Methodology

The algorithm proposed in this work enables to reduce background water losses from *WDS* by decreasing the head pressures along the links of the network to values strictly needed to allow the water distribution node by node. This objective could be achieved by deploying inside the *WDS* a specific number N_{PRV} of *PRVs* with the appropriate valve settings. Thus, once N_{PRV} has been fixed, their optimal locations and settings are evaluated by means of a Genetic Algorithm (*GA*). The location of each valve is defined by two parameters, namely, the network link and the valve's position along the link. Of course, not all of the positions are feasible, for either economical or technical reasons. Hence, a limited set of admissible positions (existing manholes, for example) must be chosen *a priori*. Moreover, for simplicity, each pipe in the *WDS* is considered eligible for *PRVs* positioning and, just for exemplification purposes, the *PRVs* are positioned in the middle of the pipes. At the end, once the optimal positions and settings of *PRVs* have been evaluated, the optimal number of valves to insert in *WDS* is evaluated by a *Cost-Benefit analysis*.

The above reported operation has been carried out considering a variable water demands during the day, following a given pattern (*EPS*).

2.1 Hydraulic Model

When two consecutive pipelines are connected by joints (hub and spigot), leakage may occur through the joints itself. Therefore, a proper hydraulic model is needed in order to perform computations. A realistic hydraulic simulator of the *WDS*, would require the modeling of each joint, connecting two consecutive pipes, as an emitter (Covelli et al. 2016). However, this would lead to an extreme *WDS* modeling with a high number of nodes even when the network is made by few links, increasing the computational efforts. Furthermore, the assignment of each emitter coefficient formula would lead to a high uncertainty. For this reason, in this work a simplified hydraulic simulator has been developed.

The water loss from joints along a link of the network can be assimilated to an uneven distributed discharge, with local discharge value depending upon the pressure value.

When discharge is evenly distributed, it can be accounted by the hydraulic network simulator by adding to the user demand at the k -th time interval of the day an initial $(1 - \alpha)(Q_{d,k})_j$ and final $\alpha(Q_{d,k})_j$ discharge to the upstream and downstream node of the j -th link (Messina 1945), where $(Q_{d,k})_j$ is the whole discharge evenly distributed along the j -th link of the *WDS* at the k -th time interval of the day, and α is a coefficient ranging between 1 and 0.

However, when leakages occur through the joints, the discharge is not evenly distributed, but the discharge values depend on the local pressures. Moreover, it is not actually continuously distributed along the link, but it consists in a sequence of concentrated leakage discharges $(q_{m,k})_j$ from N_j joints existing within the j -th link (with $m = 1, 2, \dots, N_j$), at the k -th time interval. For this reason, in this work, for the sake of simplicity, the coefficient α is set equal to 0.5 and $(Q_{d,k})_j$ is assumed to be unevenly distributed along the links with local values depending on the local pressure assumed to vary linearly between the upstream and downstream nodes of the j -th link (see Fig. 1), and it is computed as $(Q_{d,k})_j = \sum_m^{N_j} (q_{m,k})_j$, where N_j is evaluated by rounding up the ratio L_j/δ , where L_j is the length of the i -th and δ is the distance between two consecutive joints.

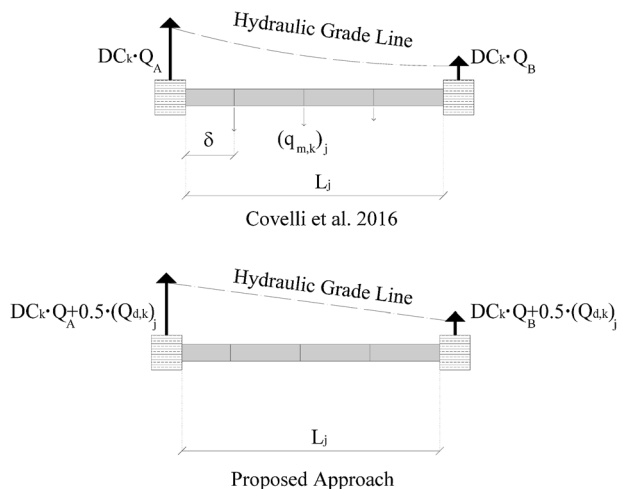
The simplified hydraulic model presented in this paper belongs to the category of pressure driven models, and allows evaluating iteratively, at each time step of the *EPS* simulation, the correct values of the leakage discharge assumed distributed along the *WDS* links.

Let N_N be the number of nodes and N_L the number of links existing in the *WDS*. Furthermore N_T be the number of time interval in which the day was divided, Q_i the daily averaged user demand at the i -th node of the *WDS*, $(Q_{d,k})_j$ the leakage discharge along the j -th link of the *WDS* at the k -th time interval, assumed to be distributed along the link with local value depending on the local pressure value and $DC_{k,k}$ the demand coefficient at the k -th time interval.

The hydraulic model presented in this work can be summarized in the following steps:

1. set $k = 1$;

Fig. 1 The evaluation of the discharges flowing out at deteriorated joints: definition sketch



2. for each link $j=1, 2, \dots, N_L$, set $(Q_{d,k})_j=0$;
3. for each node $i=1, 2, \dots, N_N$, set $Q_{i,k} = DC_k Q_i + 0.5 \sum_{j=1}^{N_{Li}} (Q_{d,k})_j$ where N_{Li} is the number of links having the upstream or downstream ending node equal to the i -th node. Then, perform the WDS simulation using any pressure driven approach (Pirozzi and Pianese 2002; Cozzolino et al. 2005);
4. for each link of the network, use formula given by Covelli et al. (2015) to evaluate $(q_{m,k})_j$, assuming that the pressure head varies linearly between the upstream and downstream ending node of the j -th link. Then sum up the leakage discharge values to obtain the new values of the distributed leakage discharges $(Q'_{d,k})_j$;
5. for each link $j=1, 2, \dots, N_L$, evaluate the error $e_j = |(Q'_{d,k})_j - (Q_{d,k})_j| / \{0.5[(Q'_{d,k})_j + (Q_{d,k})_j]\}$ (The error is relative to the average of the distributed leakage in order to avoid overflow when the distributed discharge at the previous iteration is null);
6. if $\max e_j$ is smaller than some specified tolerance, then increase k and repeat from step 2. If it is a greater than some specified tolerance, then for each $j=1, 2, \dots, N_L$ set $(Q_{d,k})_j = (Q'_{d,k})_j$ and repeat from step 3 ('feedback procedure').

The hydraulic model used in the computations is based on the software EPANET™ (Rossman 2000) and it was implemented using the EPANET™ DLL available for free.

The results obtained with the above described hydraulic model are validated by comparison with the results obtained in Covelli et al. (2016) (see the Sections 3 and 3.1).

2.2 Genetic Algorithm

The GA considered in this work was the same already proposed and used by some of authors to carry out the optimised design of urban and rural drainage networks (Cimorelli et al. 2013; Cimorelli et al. 2014; Cimorelli et al. 2016; Covelli et al. 2016; Cozzolino et al. 2015; Palumbo et al. 2014). In all case studies considered in this paper, the crossover probability p_c and mutation probability p_m were set equal 1 and 0.01 respectively. These values were set after a few trials, but they have to be chosen case by case from users. The GA maximum iteration numbers N was $N_{it}=200$ in all simulations, while the population size was $N_i=100$ in the first case study and $N_i=300$ in the second case study. Consequently, 20,000 FF and 60,000 FF evaluations were carried out for the first and for the second case study, respectively. The decision variables consist, for a pre-assigned number of pressure reduction valves N_{PRV} , of the valve positions and settings. In particular, the valve positions are identified not only by the network link but also by the direction in which the valve works.

2.3 Fitness Function

The Fitness Function (FF) conceptually consists of two parts (Covelli et al. 2016). The first one is related to the Costs (C) to be incurred and the second one consists of penalties (P) introduced when one or more constraints are not satisfied. The Costs considered in the FF are the leakage (unaccounted water that may be delivered to other users) and the costs of the valves (purchase, installation and management). The simulation period is assumed to be one day long and the users' demand pattern is a typical daily demand pattern during the year. The whole yearly amount of water saved is evaluated by multiplying the daily water volume lost, calculated by using the pressure-discharge

relationship given by Covelli et al. (2015), with the number of days present in a year. In particular, the FF is:

$$FF = C + P \tag{1}$$

where

$$C = C_{Water_Losses} + C_{PRVs} \tag{2}$$

- $C_{Water_Losses} = c_1 \cdot W_{lost} \sum_{n=1}^{N_{EWT}} (1 + r_{WL})^{n-1}$: are the costs associated with the loss of earnings as a result of the water lost from the pipes, evaluated at the end of N_{EWT} years of WDS management; N_{EWT} = the expected working time of the valves before their substitution (years); W_{lost} is the expected volume of water lost over 1 year [$m^3/year$]; c_1 is the initial cost for a single cubic meter of water lost (in these case studies this value was assumed equal to 0.818 €/m^3 , that is a typical value for the Campania Region, Italy, for the water supplied to private users); r_{WL} is the rate at which the price of the volume of water that could be delivered to users if no longer dispersed in the subsurface would grow annually;
- $C_{PRVs} = \sum_{\nu=1}^{N_{PRV}} (C_{PRV_Maint.} + C_{PRV_Inst.})_{\nu}$: are the costs related to the purchase, installation and maintenance of the selected set of $PRVs$, where: ν and N_{PRV} are the generic PRV and the whole number of $PRVs$ considered in the simulation, respectively ($\nu = 1, 2, \dots, N_{PRV}$); $C_{PRV_Maint.}$ are the *Costs* sustained for the maintenance of both the PRV and manhole; $C_{PRV_Inst.}$ are the *Costs* sustained for PRV purchasing, for the construction of related manhole and for installing the PRV in the manhole, evaluated at the end of the expected PRV working time;

and

$$P = p \cdot \sum_{k=1}^{N_T} \left\{ \sum_{i=1}^{N_{nodes}} \max \left[0, \left(H_{i,\min} - H_i^k \right) \right] \right\} \tag{3}$$

where: $H_{i,\min}$ is the ‘ideal’ value of the piezometric head at the i -th node [$m \text{ a.m.s.l.}$]; H_i^k is the piezometric head at node j during the k^{th} time step of the EPS [$m \text{ a.m.s.l.}$]; N_{nodes} is the number of delivery nodes of WDS ; p (equal to $10^{20} \text{ [€/}(m \text{ year})]$) is the ‘specific penalty coefficient’ considered to penalize, and then to exclude (asymptotically), the operating conditions for which the nodal piezometric head at delivery node i is lower than $H_{i,\min}$.

3 Applications of the Proposed Procedure

3.1 Case Studies and Discussion

An application to a first case study (*Case 1*), presented by Covelli et al. (2016), was performed in order to test the real possibilities of utilization of the optimization procedure with the

Table 1 The values of *FF* and the yearly volume of water lost from *WDS* with the number of deployed *PRVs* – *Case 1*

N_{PRV}	V_{lost} [m ³ /year]	FF [€]	Positioning [number of link]	Setting [m a.m.s.l.]
0	1231878.60	28460017.38	–	–
1	1089944.71	25228485.00	10	35.3
2	981251.39	22762387.95	18; 1	25.3; 6.9
3	963042.02	22351619.12	1; 18; 17	6.9; 25.3; 35.7
4	955254.34	22185338.86	9; 17; 10; 27	30.5; 35.8; 7.0; 24.1
5	940864.58	21868796.31	23; 27; 17; 10; 9	48.9; 24.1; 35.8; 7.0; 30.5

simplified hydraulic model described in this work. It consists of a *WDS* serving a population of 31.500 inhabitants composed of $N_L=32$ cast iron mains and $N_N=24$ nodes. Details about the geometrical data and minimal pressure head as well are reported in Covelli et al. (2016). The *WDS* delivers a daily average users demand equal to 91.15 l/s, corresponding to 70 % of water distributed each day, because the leakage discharge is supposed to be about 30 % of the total daily average discharge delivered equal to 130.21 l/s. The leaking discharges at the pipe joints were calculated by means of the approach proposed by Covelli et al. (2015), based on the utilization of a complete pressure driven approach where the water lost at each link was evaluated modelling the pipeline joints as emitters nodes (see Fig. 1), with the parameter $\xi=0.55$ calibrated to give a leakage volume about 30 % of the total daily average volume. The distance between two joints connecting two consecutive pipes constituting the links was assumed $\delta=6$ m (commercial cast iron pipes).

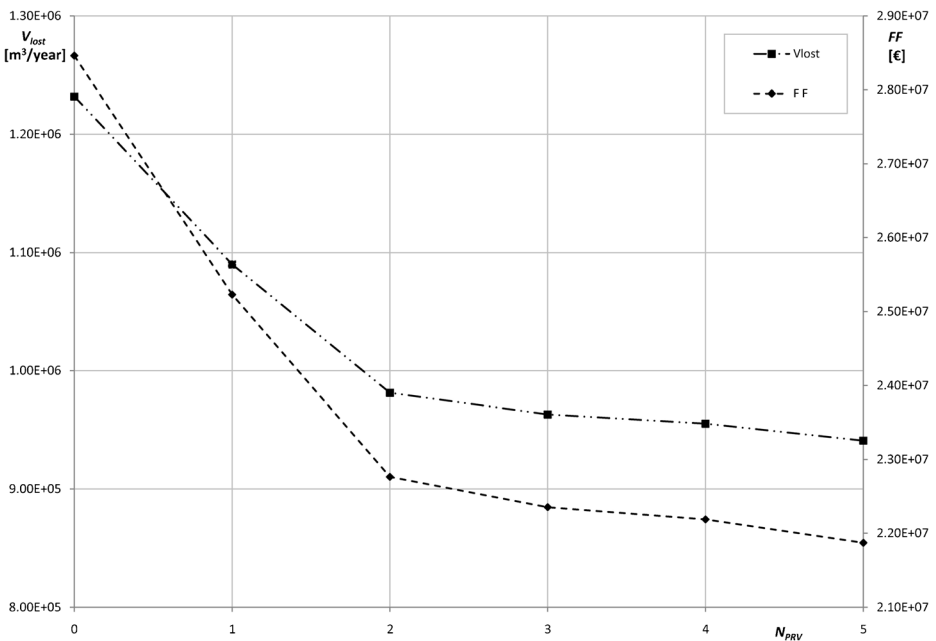


Fig. 2 The trends of the Fitness Function and the yearly volume of water lost from *WDS* with the number of deployed *PRVs*—*Case 1*

Table 2 The values of *FF* and the yearly volume of water lost from *WDS* with the number of deployed *PRV*'s obtained by proposed procedure and Covelli et al. (-2016)—*Case 1*

N_{PRV}	V_{lost} [m ³ /year]	<i>FF</i> [€]	V_{lost} [m ³ /year]	<i>FF</i> [€]
0	1231878.60	28460017.38	1231878.60	28460017.38
1	1089944.71	25228485.00	1049174.54	24294786.93
2	981251.39	22762387.95	1010678.77	23432325.61
3	963042.02	22351619.12	959448.65	22268601.71
4	955254.34	22185338.86	938367.31	21826853.18
5	940864.58	21868796.31	923858.93	21507570.25

The hydraulic simulations were carried out by means of an *EPS* approach, considering the same time pattern is reported by Covelli et al. (2016) as well as the same *PRV*'s costs for each diameter.

Five simulations were performed with a valves number going from 1 to 5. In this case study, every reach was considered a possible position of the *PRV*. Then, for each valve there are 64 possible positions: 32 position in the same direction in which the links are drawn and 32 in the opposite direction. The valve settings range from 0.0 to 5.0 atm. Results of the simulations are reported in Table 1 and the behaviour of the *FF* and the leakage volume with N_{PRV} are displayed in Fig. 2.

Comparisons between the *FF* trends obtained by using the model presented in this work and the model presented in Covelli et al. (2016) are reported in the following Table 2 and

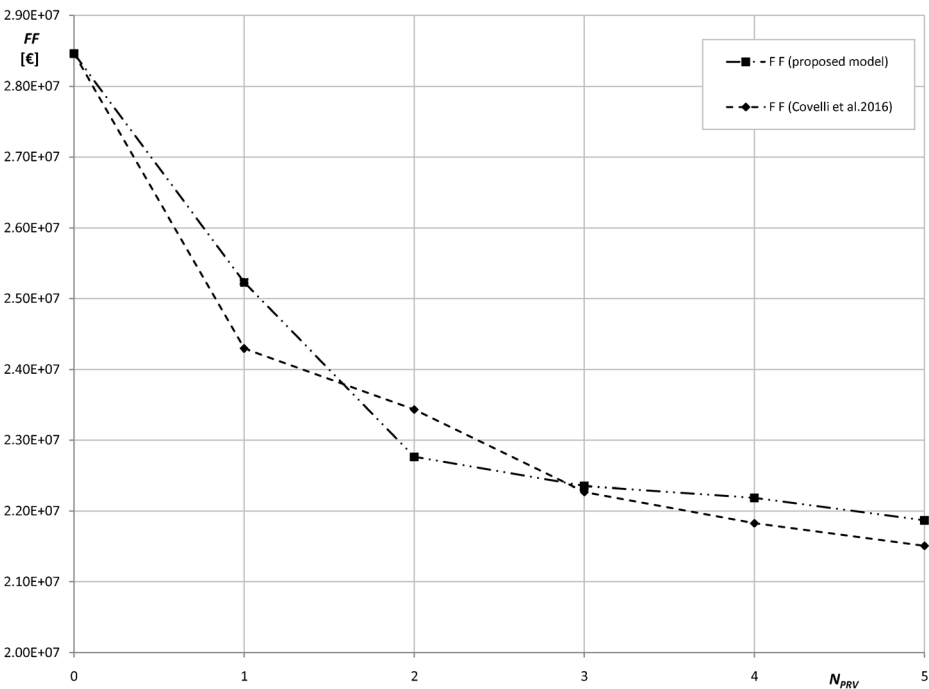


Fig. 3 A comparison of the trends of Fitness Functions variable with the number of deployed *PRV*'s between the presented methodology and Covelli et al. (2016)—*Case 1*

displayed in Fig. 3. While, comparisons of the optimal solution obtained with both models are reported in Table 3.

Although the hydraulic model considered within the optimization procedure presented in this paper is simplified, by inspection of Fig. 3, it is clear that the approach presented here is able to reproduce solution similar to that reported in Covelli et al. (2016) with a more realistic (but more time consuming) hydraulic simulator. Indeed, as one can see from Tables 2 to 3, the solutions provided by the two models are very similar in terms of both *FF* values and optimal valve positions and settings. The comparisons in terms of saved volume of both models are reported in Fig. 4.

For each number of *PRVs* considered, the percentage of saved volume evaluated by using the simplified approach proposed in this work is very close to that evaluated by using the more sophisticated approach proposed by Covelli et al. (2016). In particular, the best solutions obtained with the two models (five *PRVs*) differ only about 1 %. Then, in the case examined, the differences between the two approaches appear acceptable from the practical point of view. However, because of both the morphology of the city considered in this case study and WDS layout and diameters, for each assigned number of *PRVs*, their optimal positioning and setting is often quite different, strongly depending on the hydraulic modelling carried out within the two approaches. The strong influence of morphology and WDS layout and diameters is confirmed by the following observation: when the valve position given by the two optimization approaches is the same, the corresponding valve setting is, in turn, approximatively the same; whereas, when the valves positions given by the two optimization approaches are different, their settings congruently changes in order to attain similar piezometric patterns. For these reasons, the authors believe

For these reasons, the authors believe that the two approaches, although they can apparently lead to different results with regard to the optimum positioning and the setting of pre-assigned groups of *PRVs*, lead, in fact, almost exactly the same results in the reduction of water losses and optimal management of WDSs. In order to show the validity of the optimization procedure presented in this paper, it was also applied to a second and more complex case study (*Case 2*). The network, composed of $N_L=52$ pipes and $N_N=46$ nodes, is supplied by a tank and by pumping system. Details about the network geometry and time patterns are reported in Creaco and Pezzinga (2015). In order to compute the leakage from joints (supposed to be made of hub and spigot), the pipe geometry reported in Table 4 were hypothesized.

Table 3 The optimal positioning and setting of the assigned numbers of *PRVs* obtained by proposed procedure and Covelli et al. (2016)—*Case 1*

N_{PRV}	Positioning [number of link]	Setting [m a.m.s.l.]	Positioning [number of link]	Setting [m a.m.s.l.]
0	–	–	–	–
1	10	35.3	4	23
2	18; 1	25.3; 6.9	5; 1	36.9; 6.1
3	1; 17; 18;	6.9; 35.7; 25.3;	1; 17; 18;	6.0; 35.7; 25.3;
4	9; 17; 10; 27	30.5; 35.8; 7.0; 24.1	10; 18; 1; 27	35.3; 25.3; 6.0; 24.1
5	9; 17; 27; 23; 10	30.5; 35.8; 24.1; 48.9; 7.0;	9; 17; 27; 1; 10;	30.8; 35.8; 24.1; 6.1; 35.3

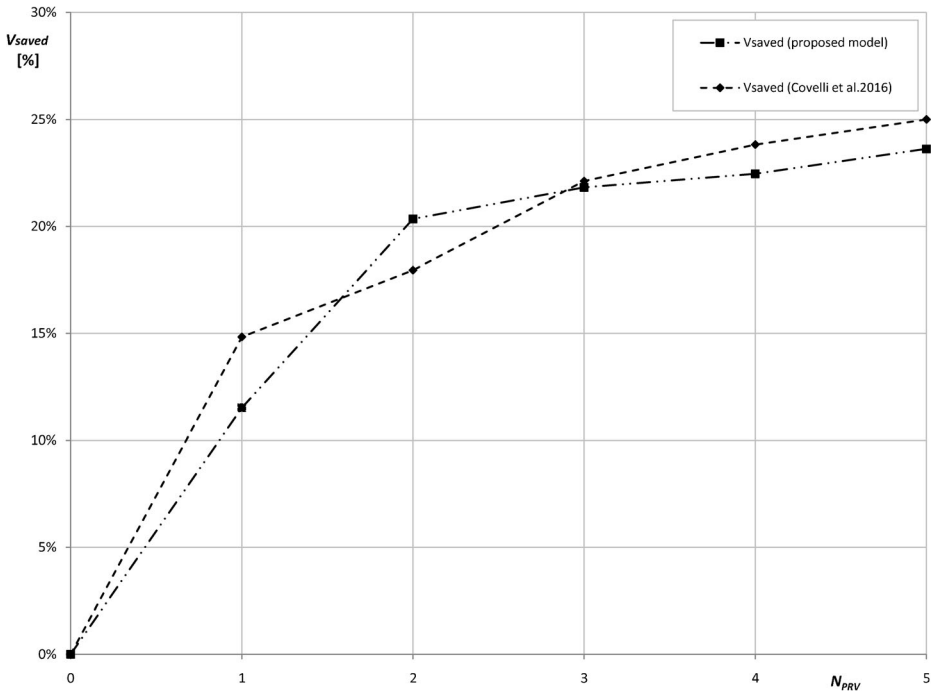


Fig. 4 A comparison of the trends of the percentage of the yearly saved volume from WDS with the number of deployed PRVs between the presented methodology and Covelli et al. (2016)—Case 1

In particular, in order to have a leakage volume about 30 % of the daily volume of water delivered, the calibration parameter of the leakage formula by Covelli et al. (2015) was found to be $\xi=0.245$. For this case study, all the intermediate points of the network links were considered as possible PRV position, except those internal to the main line (see Creaco and Pezzinga (2015)) to avoid interference with the filling and emptying process of the tank. Fifteen simulations were performed, considering an increasing assigned of PRVs, going from 1 to 15. The results of the simulations are reported in Table 5 and in Fig. 5.

In Fig. 5, the behaviour of the FF and the volume of water lost from the network are reported as a function of the number of PRVs. The optimal positioning and setting of fixed numbers of valves were found for an increasing number of PRVs, first with reference to a case

Table 4 The geometric characteristics of the pipes and related PRV's cost of Case 2

D_i [mm]	D_e [mm]	s [mm]	L_{pipe} [m]	C_{PRV} [€]
52.0	58.0	3	6	7508.58
76.0	82.0	3	6	7656.07
101.6	107.6	3	6	8029.79
102.0	108.0	3	6	8029.79
152.0	158	3	6	11632.58
203.0	209.0	3	6	15903.21
254.0	260.0	3	6	21128.77

Table 5 The values of FF, the yearly volume of water lost from WDS with the number of deployed PRVs and the optimal positioning and setting—Case 2—(* inverse flow)

N_{PRV}	V_{lost} [m ³ /year]	FF [€]	Positioning [number of link]	Setting [m a.m.s.l.]
0	285653.80	6599442.60	–	–
1	284999.67	6595971.30	25	18.6
2	280436.72	6502845.82	45; 5*	37.3; 43.5
3	276752.99	6425770.50	15; 5*; 45*	34.1; 43.3; 17.6
4	273323.91	6358181.25	45; 7*; 6*; 25	1.6; 8.3; 17.1; 18.6
5	271712.09	6320204.84	39*; 25; 5*; 32*; 6*	12.5; 18.6; 26.8; 10.6; 49.9
6	266408.09	6213343.83	38; 34*; 45; 25; 5*; 36	9.1; 12.8; 10.6; 18.6; 43.5; 7.9
7	265460.82	6203465.53	32*; 25; 7; 45; 38*; 15; 49	28.6; 18.6; 26.6; 30.4; 44.0; 34.1; 19.6
8	263850.76	6170321.72	25; 45; 5*; 37; 9; 7; 38*; 6*	18.6; c; 43.5; c; 41.1; c; 46.3; c
9	263128.20	6161284.44	15; 38*; 7; 8*; 9; 18*; 45*; 36*; 25	34.1; 48.1; 40.3; c; 20.6; 40.6; c; 32.3; 18.6
10	262309.60	6149880.88	41; 32; 17*; 15; 5*; 34; 25; 7; 37; 39*	c; c; 23.6; c; 26.8; 27.7; 18.6; c; c; c
11	257743.06	6052410.19	27; 34*; 16*; 45*; 39; 5*; 25; 28*; 6; 36; 32*	25.1; 12.8; 35.1; 17.6; 37.7; 19; 18.6; 17.6; 11.3; 12.3; 14.9
12	256405.34	6033137.52	36; 32; 39; 33; 27; 34*; 15; 18*; 6; 45*; 49; 38	c; c; c; 19.7; 25.1; 12.8; 24.1; 24.1; c; 17.6; 19.6; c
13	255897.23	6029428.51	36*; 15; 32*; 33; 17; 27*; 45*; 6; 7*; 49; 39; 5*	30.2; 25.6; 14.7; 21.1; 13.3; 50.2; 17.6; 44.6; 20.8; 1.8; 19.6; 46.6; 27.1
14	255229.88	6022788.09	15; 5; 7; 33; 49; 27*; 45*; 32*; 43; 38*; 16*; 6; 48; 39	24.1; c; 29.3; 19.7; 19.6; 10.5; 17.6; 15.5; 18.9; 44; 35.1; 17.6; 19.6; 19.8
15	254775.21	6020313.56	15; 5; 7; 33; 49; 27*; 45*; 32*; 43; 38*; 16*; 6; 48; 39; 40*	24.1; c; 29.3; 19.7; 19.6; 10.5; 17.6; 15.5; 18.9; 44; 35.1; 17.6; 19.6; 19.8; 18.7

in which all of the FF costs associated with the valves could be considered null, and then for cases in which it was necessary to consider these costs. Obviously, in the first case, the optimisation performed to search for the minimum value of FF gives the same results obtainable if the search was carried out for the minimum volume of water lost in the WDS. As a consequence, the results obtained in this particular hypothesis should be easily analysed, giving the possibility to demonstrate the ability of proposed approach to achieve the optimal solution for PRVs deployment.

In both cases, the trends for both the Fitness Function and the minimum yearly volume of water lost from the WDS seem to be very consistent with what could be hypothesised. When the number of valves N_{PRV} is increased, not only the yearly volume lost decreases but, due to the progressive redundancy of the valves introduced in the network, the rate of reduction decreases, as well. However, because of the more complex structure of the Network examined in Case 2, the trends related to Case 1 are significantly smoother than those related to Case 2. Indeed, for the second case study, a high N_{PRV} is required in order to achieve an asymptotic behaviour of the FF. This is a direct consequence of the geometrical and topological structure of the WDS considered in the second case. This is also confirmed by the inspection of Figs. 2 and 5: for Case 1, as one can see in Fig. 2, the rate at which the volume lost decreases with the increasing of N_{PRV} becomes small and stable starting from $N_{PRV}=3$, while for Case 2 it becomes small starting from $N_{PRV}=12$.

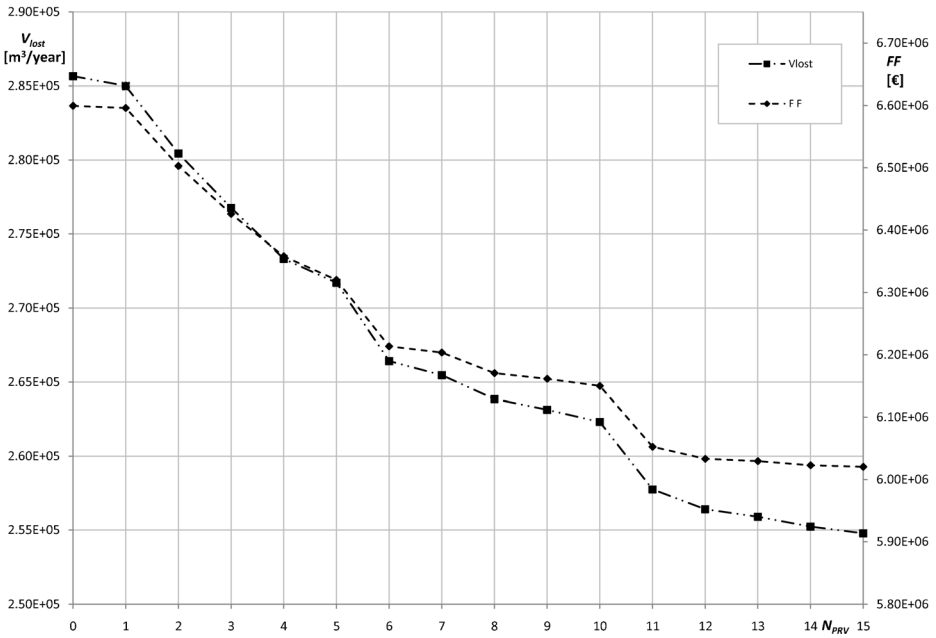


Fig. 5 The trends of the Fitness Function and the yearly volume of water lost from *WDS* with the number of deployed *PRVs*—*Case 2*

In both cases, the trend with which *FF* varies with the number of valves shows the tendency, for the increasing N_{PRV} values, to reach the maximum of the *net benefit* (defined as the difference between the economic value of the volume of water that is not more dispersed and the cost of the valves necessary to achieve the water savings).

This tendency is confirmed by the trends shown in Fig. 6, in which a cost-benefit analysis is carried out to compare the direct costs needed for *WDS* rehabilitation by the introduction of the given set of *PRVs* with the direct benefits of leakage reduction for a 25-year period.

This analysis, though does not considerate the environmental aspects, such as the greenhouse gas emissions (Venkatesh 2012), but only the economic saving, seems suitable for evaluating the optimal number of valves to use.

Figure 6 shows that the rate at which the net benefits grow with the investment costs for *PRVs* tend to decrease when the investment costs increase. Indeed, in *Case 1* when $N_{PRV}=4$, the growth rate of the net retractable benefits from the investment made for the valves tends toward zero, while for *Case 2*, it becomes negligible with $N_{PRV}=12$. Therefore, it seems not convenient to consider more than $N_{PRV}=4$ for *Case 1* and $N_{PRV}=12$ for *Case 2*.

4 Conclusions

In many cases of practical interest, the reduction of leakage in *WDSs* can be obtained by reducing the pressure through the system: this, in turn, leads to the reduction of the costs associated with the loss of water. Special devices, such as the *PRVs*, can be used to regulate the pressure, but their use introduces additional costs due to the implementation and operation of the valves. In this paper, a procedure for the optimal positioning and setting of increasing sets

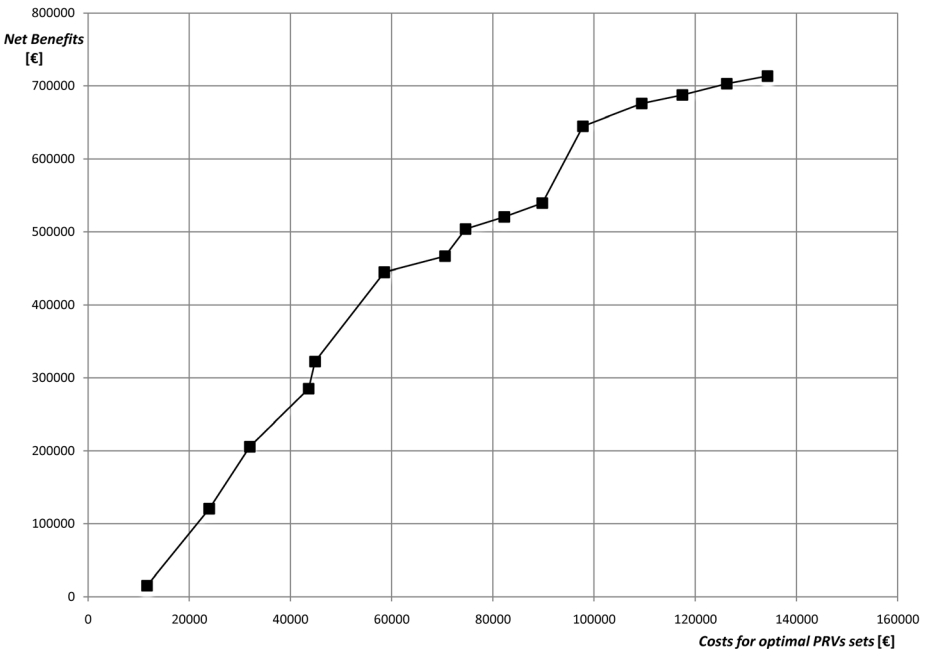
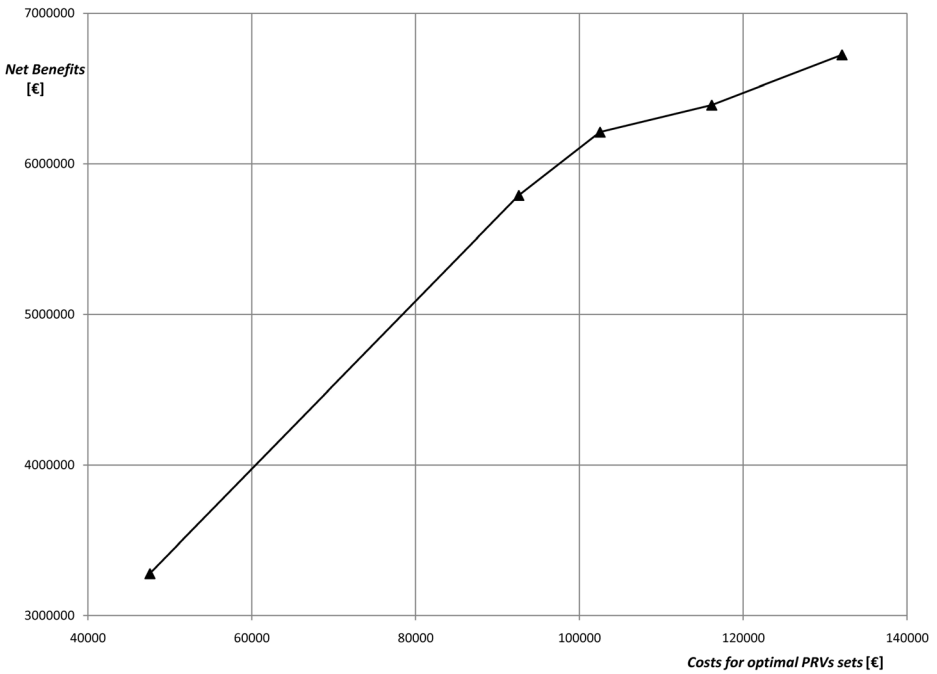


Fig. 6 The Cost-Benefit curves—Case 1 and Case 2

of valves coupled with a simplified hydraulic modelling of pressure dependent leakages through links is presented. The procedure was first demonstrated by using a realistic small

network, and comparing the results with those obtained by another optimization procedure that uses a more complicated hydraulic simulator, and then applied to a more complex water distribution system.

It is well known that the reduction of water leakage leads to energy savings in pumped systems, and the pressure management actions performed lead to cost savings, which arise from the reduced rate of pipe breakage. The approach proposed in the present work, even though these sources of financial savings are not explicitly taken into account, is completely general. Thus, to minimise simultaneously the water loss and the energy and pipe rehabilitation costs, the economic analysis framework considered in this paper can be easily enriched by using a multi-objective function to consider these additional beneficial effects.

Acknowledgments The present work was developed with financial contributions from the Campania Region, L.R. n.5/2002 – year 2008 – within the Project ‘Methods for the evaluation of security of pressurized water supply and distribution systems towards water contamination, also intentional, to guarantee to the users, and the optimization design of water systems’, prot. 2014.0293987 dated 29.04.2014 – CUP: E66D08000060002.

References

- Abdelmeguid H (2011) Pressure, leakage and energy management in water distribution systems. PhD. Thesis, De Montfort University, Faculty of Technology, Leicester, UK, p 240
- Abdelmeguid H, Skworcow P, Ulanicki B (2011) Mathematical modelling of a hydraulic controller for PRV flow modulation. *J Hydroinf* 13(3):374–389
- Araujo LS, Ramos H, Coelho ST (2006) Pressure control for leakage minimisation in water distribution systems management. *Water Resour Manag* 20(1):133–149
- Cimorelli L, Cozzolino L, Covelli C, Mucherino C, Palumbo A, Pianese D (2013) Optimal design of rural drainage networks. *ASCE J Irrig Drain Eng* 139(2):137–144
- Cimorelli L, Cozzolino L, Covelli C, Della Morte R, Pianese D (2014) Enhancing the efficiency of the automatic design of rural drainage networks. *J Irrig Drain Eng* 140(6), 04014015
- Cimorelli L, Morlando F, Cozzolino L, Covelli C, Della Morte R, Pianese D (2016) Optimal positioning and sizing of detention tanks within urban drainage networks. *J Irrig Drain Eng* 142(1), 04015028
- Covelli C, Cozzolino L, Cimorelli L, Della Morte R, Pianese D (2015) A model to simulate leakage through joints in water distribution systems. *Water Sci Technol Water Supply* 15(4):852–863. doi:10.2166/ws.2015.043, IWA Publishing
- Covelli C, Cimorelli L, Cozzolino L, Della Morte R, Pianese D (2016) Reduction in water losses in water distribution systems using PRVs, *Water Science and Technology: Water Supply*. doi:10.2166/ws.2016.020
- Cozzolino L, Covelli C, Mucherino C, Pianese D (2005) Hydraulic reliability of pressurized water distribution networks for on-demand irrigation. In: Savic D, Walters G, Khu ST, King R (eds) *Proceedings of the Eight International Conference on Computing and Control for the Water Industry CCWI 2005, Water Management for the 21st century*, vol 1. Centre for Water Systems, University of Exeter, Exeter, pp 91–97, ISBN: 0-9539140-3-8
- Cozzolino L, Cimorelli L, Covelli C, Mucherino C, Pianese D (2015) An innovative approach for drainage network sizing. *Water* 7(2):546–567
- Creaco E, Pezzinga G (2015) Embedding linear programming in multi objective genetic algorithms for reducing the size of the search space with application to leakage minimization in water distribution networks. *Environ Model Softw* 69:308–318
- Dai PD, Li P (2016) Optimal pressure regulation in water distribution systems based on an extended model for pressure reducing valves. *Water Resour Manag* 30(3):1239–1254
- Germanopoulos G (1995) Valve control regulation for reducing leakage. In: Cabrera E, Vela AF (eds) *Proc. improving efficiency and reliability in water distribution systems*. Volume 14 of the series *Water Science and Technology Library*, pp 165–188
- Germanopoulos G, Jowitt PW (1989) Leakage reduction by excess pressure minimization in water supply networks. *Proceedings - Institution of Civil Engineers. Part 2. Research and theory* 2(87):195–214

- Hunaidi O (2010) Leakage management for water distribution infrastructure – report 1: results of DMA experiments in Regina, SK, Ottawa, Canada. National Research Council
- Jowitt PW, Xu C (1990) Optimal valve control in water distribution networks. *J Water Resour Plan Man ASCE* 116(4):455–472
- Lambert A, Thornton J (2011) The relationships between pressure and bursts - a ‘state-of-the-art’ update, *IWA Water 21 Journal*
- Liberatore S, Sechi GM (2009) Location and calibration of valves in water distribution networks using a scatter-search meta-heuristic approach. *Water Resour Manag* 23:1479–1495
- Marunga A, Hoko Z, Kaseke E (2006) Pressure management as a leakage reduction and water demand management tool: the case of the City of Mutare, Zimbabwe. *Phys Chem Earth* 31:763–770
- Messina U (1945) Metodi approssimati per i calcoli di verifica delle reti di condotte idrauliche. *L’Acqua* 1(12), pp 1–11 (in Italian)
- Nicolini M, Zovatto L (2009) Optimal location and control of pressure reducing valves in water networks. *J Water Resour Plan Manag* 135:178–187
- Palumbo A, Cimorelli L, Covelli C, Cozzolino L, Mucherino C, Pianese D (2014) Optimal design of urban drainage networks. *Civ Eng Environ Syst* 31(1):79–96
- Pirozzi F, Pianese D, D’Antonio G (2002) Water quality decay modelling in hydraulic pressure systems. *Water Sci Technol Water Supply* 2(4):111–118
- Prescott SL, Ulanicki B (2008) Improved control of pressure reducing valves in water distribution networks. *J Hydraul Eng ASCE* 134(1):56–65
- Puust R, Kapelan Z, Savic DA, Koppel T (2010) A review of methods for leakage management in pipe networks. *Urban Water J* 7(1):25–45
- Ramos H, Mello M, De P (2010) Clean power in water supply systems as a sustainable solution: from planning to practical implementation. *Water Sci Technol Water Supply* 10(1):39–49, IWA Publishing
- Reis LFR, Porto RM, Chaudhry FH (1997) Optimal location of control valves in pipe networks by genetic algorithm. *J Water Resour Plan Manag ASCE* 123:317–326
- Report n. 26 Leakage Control Policy and Practice, Technical Working Group on Waste of Water (1985) UK Water Authorities Association, WRc, ISBN 094561 95 X
- Rossman LA (2000) EPANET 2 user’s manual. U.S. EPA, Cincinnati
- Savić D, Walters GA (1996) Integration of a model for hydraulic analysis of water distribution networks with an evolution program for pressure regulation. *Microcomput Civ Eng* 11:87–97
- Tabesh M, Hoomehr S (2007) Consumption management in water distribution systems by optimizing pressure reducing valves’ settings using genetic algorithm. *Desalin Water Treat* 2:96–102
- Thornton J (2003) Managing leakage by managing pressure: a practical approach. *Water 21*, magazine of the International Water Association. October 2003, pp 43–44
- Tucciarelli T, Criminisi A, Termini D (1999) Leak Analysis in pipeline systems by means of optimal valve regulation. *J Hydraul Eng* 125(2–3):277–285
- Ulanicki B, Bounds PLM, Rance JP, Reynolds L (2000) Open and closed loop pressure control for leakage reduction. *Urban Water J* 2:105–114
- Ulanicki B, AbdelMeguid H, Bounds P, Patel R (2008) Pressure control in district metering areas with boundary and internal pressure reducing valves. Proceedings of 10th International Water Distribution System Analysis conference, WDSA2008, The Kruger National Park, Cape Town, South Africa. doi:10.1061/41024(340)58
- Vairavamorthy K, Lumbers J (1998) Leakage reduction in water distribution systems: optimal valve control. *J Hydraul Eng ASCE* 124:1146–1154
- van Zyl JE, Clayton CRI (2005) The effect of pressure on leakage in water distribution systems. In: Savic D, Walters G, Khu ST, King R (eds) Proceedings of CCWI2005 Conference. Exeter, UK, vol 2, pp 131–135
- Venkatesh G (2012) Cost-Benefit analysis–leakage reduction by rehabilitating old water pipelines: case study of Oslo (Norway). *Urban Water J* 9(4):277–286
- Walski T, Bezts W, Posluszny ET, Weir M, Withnam BE (2006) Modeling leakage reduction through pressure control. *J AWWA* 98(4):147–155
- Xu Q, Chen Q, Ma J, Blanckaert K, Wan Z (2014) Water saving and energy reduction through pressure management in urban water distribution networks. *Water Resour Manag* 28(11):3715–3726