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Operational and Tactical Management of Water and Energy Resources in Pressurized Systems: the competition at WDSA 2014

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ABSTRACT

Optimal management of water and energy resources worldwide is a basis for environmental and socio-economic sustainability in urban areas, which has become even more relevant with the advent of the "smart" and "water sensitive" city paradigm. In water distribution networks (WDNs) water resource management is concerned with increased efficiency, which is mainly related to the reduction of leakages, while energy management refers to optimal pump, valve and source scheduling strategies considering the hydraulic system requirements. These management goals require planning of asset renewal and improvement works in the short time (operational) and medium time (tactical) horizons, considering the financial sustainability of relevant actions. The Battle of Background Leakage Assessment for Water

Networks (BBLAWN) was designed as a competition held at the 16th Water Distribution Systems Analysis Conference, in Bari (Italy) in 2014 (WDSA 2014), to address the aforementioned management goals. The teams taking part in the BBLAWN were asked to develop a methodology for both reducing real water losses and saving energy in a real WDN considering the possibility of asset renewal and strengthening. Fourteen teams from academia, research centers and industry presented their solutions at a special session of the WDSA 2014 conference. This paper briefly describes the BBLAWN and presents one of the solutions provided by the organizers to illustrate the ideas and challenges embedded in the posed problem.

The overview of the solutions provided by the participants shows that management decisions need to be supported by engineering judgment as well as with tools that combine computationally effective multi-objective optimization and hydraulic models capable of assessing pressure-dependent background leakages.

Keywords

Water distribution network; Integrated management; Leakages; Energy; Hydraulic models; Pressure Reduction Valves.

Introduction

The series of “Battle Competitions” date back to 1985 with the Battle of the Water Networks (BWN) (Walski et al., 1987), and was created to stimulate academia, research centers and industry to provide solutions and strategies for addressing complex practical problems in water distribution network (WDN) analysis, design and management. More recently the Battle of the Water Sensor Networks (BWSN) (Ostfeld et al., 2008) was held in 2006 in Cincinnati (OH, USA); the Battle of the Water Calibration Networks (BWCN) (Ostfeld et al., 2012) was held in 2010 in Tucson (AZ, USA); the Battle of the Water Networks Design (BWN-II) (Marchi et al., 2014) was held in 2012 in Adelaide (Australia).

The Battle of Background Leakage Assessment for Water Networks (BBLAWN) was held at the 16th Water Distribution Systems Analysis Conference, in Bari (Italy), in July 2014 (WDSA 2014), thus being the fifth "Battle" on WDNs. The problem was designed to stimulate a discussion about the optimal management of water and energy resources in WDNs. This is actually an emerging issue relevant from environmental and socio-economic perspective worldwide, also pertaining to smart city paradigm.

The complexity of WDN analysis and management is increasing due to the growth of population in urban areas and the increase of system size and interconnectivity. Real water losses in Europe range from 20 to 40% (and more in some environments) mainly due to the natural asset deterioration of aged WDNs. For example, in Italy, it is estimated that over 64 billion euros will be needed in the next 30 years for WDN rehabilitation (FederUtility's Blue Book 2011). The asset deterioration and the consequent real water losses are relevant water management issues because the inefficient use of water resources exacerbates the impact of water scarcity due to socio-economic factors and/or climate changes. Therefore, water companies seek management solutions and convincing/effective decision making strategies to support real leakages reduction in short-medium and long time horizons and for managing the rapid deterioration of assets which has an enormous public value. These facts make urgent for water utilities to undertake actions in the short-medium time horizon, which need to be effective also in the long time horizon. Optimal management of water resources in WDN actually means to minimize water losses from deteriorated infrastructures and, more explicitly, the background leakages from pipes. These type of distributed losses are less evident than major bursts and usually run for longer before repair (Germanopoulos, 1985). In addition, in aged pipes the joint effect of both increased head losses (due to increased internal roughness) and background leakages causes pressure drop through the system. A commonly adopted countermeasure for this consists of increasing water pumping into the system in order to provide sufficient pressure to deliver water to a service reservoir or directly into distribution. This, in turn, results in increased water losses and energy consumption.

Thus, water and energy management are directly related and depend on WDN operation (e.g. filling/emptying of tanks), pressure regime through the network and the total water demand, including both customers' water requirements and leakages (Giustolisi and Walski, 2012).

On this premise, minimizing water and energy consumption is a complex problem that, in the short-term horizon, requires effective *operational* strategies, as well as sustainable asset renewal plans for the *tactical* planning (medium term horizon). In fact, the reduction of water leakages in the short time horizon could be achievable by implementing optimal pumping (e.g. Giustolisi et al., 2013) as well as by installing pressure control valves to avoid excessive pressure in some parts of the network. Nonetheless, in real systems there is a range of technical asset management options including pipe renewal (e.g. replacement, relining) or installation of new pipes in parallel to the existing ones, enlargement of existing tanks or enhancement of pumping stations. The selection of the most effective alternative needs to be evaluated in the medium term horizon, and in conjunction with optimal operation strategies. In addition, each technically feasible solution needs to be evaluated in terms of financial sustainability, considering total costs, i.e., both operational (OPEX) and capital (CAPEX) expenditure, in order to be readily evaluated by water utilities.

The Battle of Background Leakage Assessment for Water Networks - BBLAWN

The BBLAWN called for teams from academia and industry to design a methodology for reducing water losses due to background leakages, considering the cost for upgrading the hydraulic system capacity. The intervention options available to the teams were pipe replacement or installation in parallel to existing pipes, installation of new parallel pumps and enlarging tanks (i.e. addition of new cylindrical tanks adjacent to the existing tanks), the installation of pressure control valves (PRVs), while considering also the cost of energy and water losses (see Giustolisi et al. (2014) and BBLAWN webpage for further details). The aim was to stimulate competing teams to deal with the conflicting cost objectives (i.e., asset upgrading versus energy cost and leakage reduction versus system pressure reduction using costly control valves).

Actually, devising strategies for water leakage management should encompass also environmental and social sustainability criteria, beyond economic and technical objectives. The BBLAWN problem statement accounts for such aspects in terms of “externalities” representing environmental and social costs and benefits (Delado-Galvan et al., 2010; European Commission, 2013) like, for example, the impact on water resources or the damages caused by leakages. The externalities are computed using the cost of water as a proxy for the environmental and resource cost, beyond the operational costs (that are part of the water tariff for customers and is related to the water company annual balance). Based on these considerations, the cost of water lost volume is fixed at 2 €/m^3.

The competition used C-Town (Ostfeld et al. 2012) whose network layout is reported in Fig. 1. To solve the BBLAWN problem, it was assumed that the city has already commissioned the development of a calibrated hydraulic model of the existing network to be used in evaluating its present state and future improvements and performance. Therefore, the network model includes the network layout, the demand patterns and the background leakage model parameters. It also contains existing pump and tank characteristics and the controls of pumps and valves based on water level in tanks.

The existing infrastructure is not able to meet the pressure performance target of 20 m at each node with demand, and the situation is compounded by excessive background leakage. Therefore, the water utility is interested in minimizing operational and capital costs.

The design problem must be solved as a one-stage intervention problem (i.e. both operational and capital costs to be minimized are reported as annual cost, which account for the lifetime of the single component and the discount rate), and the teams were asked to come up with a solution respecting other common engineering considerations and operational constraints in order to propose a methodology and provide one feasible solution from the utility standpoint. For this reason, the solutions were evaluated by the organizers in terms of operational and capital costs, but also accounting for the engineering soundness of the methodology used and the technical justification for the choices taken by the teams (e.g. the consistency with the expected hydraulic WDN behavior over an operational cycle, the rationale behind the selection of candidate pipes for replacement or the location of PRV, and so on).

In fact, the BBLAWN competition was designed as close as possible to a real situation in terms of complexity and design/operational options. This was aimed at stimulating the discussion and exchange of information among the different teams about the use of optimization tools, the need for enhanced hydraulic modelling to predict the background leakages and the whole system behavior, as will be discussed in the next section.

Fig. 1. TOWN-C for BBLAWN composed of 444 pipes, a reservoir (R), seven tanks (Tx), eleven pumps (PM), a control valve (CV), a check valve (CH).

Hydraulic and Leakage Modelling

Water leakage is caused by small or large breaks and openings in pipes, which occur at water mains and along the pipe connections to properties. The technical literature classifies leakages in background and burst leakage (unreported or reported) depending on the level of outflow. Germanopoulos (1985) proposed the following model for background leakages:

$$d_k^{leaks}\left(P_{k,mean}\right)=\begin{cases}\beta_k L_k P_{k,mean}^{\alpha_k} & P_{k,mean}>0\\ 0 & P_{k,mean}<0\end{cases} \tag{1}$$

where k = index referring to the kth pipe; $P_{k,mean}$ = model mean pressure along the kth pipe in [m] (see next section for details); d_k^{leaks} = background leakages outflow along the kth pipe in [m^3/sec]; α_k [-] and β_k [$m^{2-\alpha}$/s] = model parameters; L_k = length of the kth pipe, in [m].

Background leakages are diffuse (spatially distributed) and low intensity losses (outflows) along pipes (mains and connections), which depend on the asset condition, i.e., as related to the multiplier β in Eq. (1). They run continuously over time and could cause significant losses from the system.

Bursts are the natural evolution of background leakages due to external forces/factors, which act on deteriorated pipes. The model in Eq. (1) is aimed at predicting the outflows of diffuse leakages, considering also unreported small bursts, thus it is useful for planning purposes. This is opposite to burst modelling, which is much more suited for operational purposes, e.g., for outflow location and consequence prediction. Therefore, the competing teams were asked to employ hydraulic modelling considering background leakages (Giustolisi et al., 2008) because the hydraulic consistent prediction of those outflows not only influences the computation of the water losses but also the assessment of the system capacity, energy and water use.

The need for an accurate prediction of the system behavior is important to design an effective solution for real systems. To this purpose, the teams were asked to compute the energy for pumping using the following formulations involving expressions for variable head and efficiency:

$$\begin{aligned} &H = H^s - rQ^c \\ &\eta = -\frac{4\eta_{max}}{Q_{max}^2}Q^2 + \frac{4\eta_{max}}{Q_{max}}Q \\ &Q_{max} = \left(\frac{H^s}{r}\right)^{\frac{1}{c}} \end{aligned} \tag{2}$$

where η_{max} = maximum pump efficiency; H^s, r and c = parameters of the pumps. Eq. (1) represents a parabolic function with the maximum value (η_{max}) at $Q_{max}/2$ (Giustolisi et al., 2013).

Background Leakages versus Burst Modelling

Background leakage modelling, Eq. (1), for planning purposes is different from modelling a single burst for operational purposes like, for example, for its detection and/or preliminary localization.

The model in Eq. (1) depends on the average pressure in pipes, because leakages along mains and pipe connections are dependent on pressure. Consequently, the average local pressure is a good indicator influencing the total leakage in a pipe. In fact, the model in Eq. (1) states that the overall leakage outflow (the volume of water losses), is proportional to the average, i.e. local, pressure in the hydraulic system where the exponent α is related to the pipe material (i.e. stiffness) (Giustolisi et al., 2008). From the hydraulic modelling point of view, it is important to remark that, given the *k*-th pipe whose end nodes are *i* and *j*, the model for background leakages in Eq. (1) is different from the model for pipe bursts (i.e., outflows from nodes). The model in Eq. (1) states that the background leakages for pipe *k* are:

$$d_k^{leaks}\left(P_{k,mean}\right) = \beta_k L_k \left(\frac{P_i + P_j}{2}\right)^{\alpha} \tag{3}$$

and, for modelling purpose, such background leakage outflow along the *k*th pipe is concentrated at two water withdrawal points at the end nodes, and divided equally:

$$d_i^{leaks}\left(P_{k,mean}\right) = d_j^{leaks}\left(P_{k,mean}\right) = \frac{\beta_k L_k}{2}\left(\frac{P_i + P_j}{2}\right)^{\alpha} \tag{4}$$

Lumping the pipe level outflow at the end nodes preserves the mass balance while causes an error in the energy balance equation. The magnitude of the error can be evaluated as in Giustolisi and Todini, (2009) and Giustolisi (2010).

The strategy of using a concentrated outflows at pipe ending nodes characterized by the outflow coefficient $\beta_k L_k/2$, (i.e., assuming a burst model surrogating the background leakage model), results in the following computed outflows from nodes *i* and *j* respectively:

$$d_i^{leaks}(P_i)=\frac{\beta_k L_k}{2}(P_i)^\alpha;\; d_j^{leaks}(P_j)=\frac{\beta_k L_k}{2}(P_j)^\alpha \quad \Rightarrow$$
$$d_k^{leaks}(P_i,P_j)=\beta_k L_k \frac{(P_i)^\alpha+(P_j)^\alpha}{2} \tag{5}$$

This assumption generates a modelling error, represented by the difference between d_k^{leaks} of Eq. (5) and Eq. (4), that is actually a function of asset (i.e. α, β, L) and nodal pressures,

$$\beta_k L_k \left(\frac{P_i+P_j}{2}\right)^\alpha - \beta_k L_k \frac{(P_i)^\alpha+(P_j)^\alpha}{2} = f\left(\alpha,\beta_k,L_k,\left|P_i-P_j\right|\right) \tag{6}$$

It is worth noting that nodal outflows computed by Eq. (4) and (5) are different even if α =1 is used:

$$\begin{array}{lll} d_i^{leaks}=\dfrac{\beta_k L_k}{2}\left(\dfrac{P_i+P_j}{2}\right) & d_j^{leaks}=\dfrac{\beta_k L_k}{2}\left(\dfrac{P_i+P_j}{2}\right) & \text{background leakage model} \\ d_i^{leaks}=\dfrac{\beta_k L_k}{2}P_i & d_j^{leaks}=\dfrac{\beta_k L_k}{2}P_j & \text{burst model} \end{array} \tag{7}$$

Indeed, Eqs (4) and (5) return different leakage outflows lumped at nodes causing different pressures through the network, which, in turns, change the background leakage outflows.

In summary, for any $\alpha \neq 1$, the difference between the background leakages prediction on a single pipe is evident as reported in Eq. (6), while for α=1 the predictions become different because the demands and pressure distribution in the network are different.

Solution of the Competition Organizers

The organizers of the BBLAWN also solved the problem in order to verify its feasibility and provide a further contribution to the discussion. The solution is developed using a mix of engineering judgment, system optimization and extended period simulation (EPS) analysis aimed at supporting the decisions step by step. The solution was designed in three steps that are summarized here and detailed in the following.

Step 1. Pump scheduling optimization of the original hydraulic system is performed first without upgrading any assets. The step is useful for the assessment of the initial level of leakage (assuming optimal pumping) and the hydraulic capacity of the system. The EPS analysis of the optimized system allowed the identification of critical nodes in terms of pressure requirements. Together with the analysis of the hydraulic behavior of the WDN they were used to select candidate pipes for replacement in the comprehensive system optimization of step 2.

Step 2. Hydraulic system optimization is performed considering the cost of: (i) pipe replacement; (ii) tank enlargement; (iii) new installed parallel pumps; (iv) pump scheduling; and (v) water loss reduction. Before optimization runs, some pipes of the WDN were closed at no cost (since in the BBLAWN problem statement an isolation valve is assumed present on each pipe; these pipes are reported as dotted lines in Figure 2). Indeed, closing a pipe allowed all the water feeding a network segment to go through the pipes with a PRV. It was assumed that PRVs are not installed yet in Step 2 but they would be installed in the future with the option of a multi-stage intervention strategy.

Step 3. Pump scheduling optimization is performed by considering 25 PRVs already installed, and the asset-intervention solution obtained in step 2. The pump scheduling problem was then solved and the 25 PRVs were ranked based on their individual contribution to the reduction of water losses. On the one hand, this strategy permitted to have the total cost of the intervention together with the total expected reduction of energy and water loss costs (as requested by BBLAWN rules). On the other hand, it supports the utility in selecting the most effective sequence of valves to install considering the incoming of budget and the marginal advantage of each installation.

Step 1.Optimal pump scheduling of the original hydraulic system

This stage provided a solution showing a small pressure deficit at two nodes (indicated with empty black circles) in Figure 2, occurring at the first hour of the weekly operational cycle. The volume of water losses during the week was 36,281 m^3, corresponding to 26.05 % of the total water put into the system, which corresponds to the weekly customer demand of 102,973 m^3. The weekly energy consumption was 42,221 KWh, corresponding to a cost (given the energy tariff pattern) of 5,176 €. The solution of this stage was helpful for understanding WDN behavior over time (EPS analyses). In addition, it represents the maximum system performances achievable without any asset upgrade, thus being of direct relevance for the water utility.

Step 2. Hydraulic system optimization with upgrade of hydraulic capacity and closing pipes

The engineering judgment and EPS analyses drove the system optimization mainly to upgrade the system hydraulic capacity. To this purpose, the candidate pipes to be replaced were identified as those located along the transmission lines (see blue segments in Figure 2). There are three basic motivations for selecting the main transmission pipes.

1. The hydraulic capacity of the network was reduced by closing some additional pipes (dotted lines in Figure 2) to prepare the system for the installation of PRVs (based on engineering judgment). This affected the ability to deliver water from the pump system of DMA 1 (i.e., close to the reservoir) to the tanks n.2 and n.6 (see Figure 1) and to the four inline pump systems of DMAs 2-5.
2. As it is not hydraulically feasible to reduce the pressure along transmission pipes by installing PRVs, it is better to replace these pipes in order to reduce the volume of water losses. In addition, from system reliability perspective is better to renew transmission pipes whose failure would

reduce significantly the hydraulic capacity.

3. Interventions on transmission pipes are cost efficient for the utility considering a one-stage intervention. Furthermore, this approach reduced the search space during the optimization stage, which improved in terms of computational efficiency and effectiveness.

Consistently with the choice of increasing the system hydraulic capacity, six new parallel pumps were assumed as candidates for upgrading the pump system of DMA 1 and two for each inline pump systems of DMAs 2-5. Finally, tanks were considered as candidate for enlargement in order to reduce the energy cost (through optimal pumping) and to increase the hydraulic capacity of the DMAs 2-5, where pipes were not replaced, together with the possibility to increase the maximum power of the local pump systems.

Fig. 2. TOWN-C pressure control valve (PRV) and node of pressure set (Pset).

In summary, the overall approach was to divide the network into smaller areas by using 25 PRVs (whose settings will be defined in step 3) in order to reduce the pressure locally and increase the hydraulic capacity by means of the replacements of DMA 1 transmission pipes. Additionally, upgrading the main pump system (in DMA1) and tank n.2 was also considered. Furthermore, it is possible to increase the local hydraulic capacity of the DMAs 2-5 by upgrading inline pump systems and by enlarging internal tanks.

Figure 3 shows only the capital costs of Pareto solutions obtained by the multi-objective optimization procedure, where separate costs (i.e., pipe and pump cost; energy and water loss cost; and tank enlarging cost) were minimized simultaneously. This was achieved by using a dedicated function available in the WDNetXL system (Giustolisi et al., 2011) that permits to manage the entire problem using advanced hydraulic simulation (e.g. accounting for pressure-driven simulation of background leakages and remotely controlled PRV) and decision support (e.g. for optimal pump scheduling and asset upgrades) functions developed in the latest technical-scientific research working in Microsoft-Excel® environment (for details www.hydroinformatics.it). It is worth recalling that Figure 3 refers to the capital costs only, since the main aim of step 2 is to support decisions on asset upgrade. The fifth solution from the left of the Pareto front (see Figure 3) was selected based on engineering judgment. Indeed, this solution permits the WDN hydraulic capacity to increase by replacing seven pipes and enlarging two tanks, with tank water levels controlling the pumps. This entails cheap asset strengthening works, which could be immediately implemented by the water utility, being also a good starting point for next optimizations. The solutions results in 25.11 % of leakages and required 13,306 € for the replacement of pipes and 44,660 € for the enlargement of tanks T2 and T3 to the maximum volume of 1,693 m^3 and 180 m^3, respectively.

Figure 2 reports a black solid circle on the seven replaced pipes of the transmission line and a square on the enlarged tanks (i.e. T2 and T3). A pipe was also replaced (based on EPS analysis) in one segment of DMA 1 that was prepared to allocate a PRV (indicated with “7” in Figure 2) (by closing two pipes). Finally, the solution has one new pump (identified with a white square in Figure 2), at the cost of 4,339 €, to be installed for the DMA 2. The total weekly energy consumption for this solutions is 42,164 KWh, corresponding to a cost (given the energy tariff pattern) of 5,074 €.

Fig. 3. Pareto front of solutions for the multi-objective optimization problem (pipe and pump cost vs. energy and water loss cost vs. tank enlarging cost).

Step 3. Pumping optimization considering all the PRVs and ranking of their installation

Once the upgrading of assets was completed, the EPS analysis was performed to locate critical nodes for controlling PRVs. Remotely controlled pressure devices were used and critical nodes were selected based on the elevation and the hydraulic distance from the valves (remote set control points of PRVs are reported as red triangles in Figure 2). The selection of the critical nodes in a DMA (i.e., experiencing minimum pressure) to control PRVs allows setting the pressure at 20 m (minimum pressure for a correct service) which does not change over time (Giustolisi and Walski, 2012). This way the optimal control of the degree of valve opening does not require modulating the pressure based on the node immediately downstream from the PRV, which needs to be predicted by the model based on assumptions about demand variation over time. Of course, such solution requires that the hydraulic model to be used for assessing system performances is capable of simulating remotely controlled PRVs.

Furthermore, the pressure in the segment with no demand (see shadowed area in Figure 2) was kept low by setting it at 2 m at the critical node (i.e. as per BBLAWN rules). The pumping schedules with the setting of 25 PRVs was then optimized achieving a solution with the 18.60% of leakages (23,531 m^3 of water loss) and 37,430 KWh of energy consumption corresponding to a reduced cost of 4,438 €.

The above optimal pumping schedule was set and the EPS analysis was performed assuming the installation of one PRV at a time. The 25 PRVs were ranked in descending order based on leakage of reduction achievable by installing each PRVs. This was followed by analyzing the cumulative effect of the sequential installation of 25 PRVs. Table 1 reports the results in terms of weekly water losses, percentage of leakages and energy consumption expected by progressively adding PRVs.

Table 1 could be used as a multi-stage intervention support system allowing the user to assess the residual water losses and energy reduction. It is possible to optimize pumping for each new installation as the control of pumps by tank levels is robust with respect to small variations of demand and/or leakages (Giustolisi et. al, 2014). Finally, Table 2 summarizes the relevant data considering the original and the optimized solutions.

Table 1. Ranking of the PRVs

Table 2. Relevant data of the initial and final status of the network. Operational costs are weekly-based.

The solution obtained by organizers has an annualized capital cost of 62,305 € + 26,182 € (i.e. for the investment upgrading the asset and for the installation of PRVs), while the reduction of the weekly-based operational costs with respect to the initial condition is about 26,000 € (although that cost is not merely based on economic evaluations regarding the water losses but also financial consideration, as it accounts for the savings achievable as PRVs are progressively installed). If the cost of the lost water was assumed to be 0.5 €/m^3, thus neglecting “externalities” in the water cost (e.g., the impact of socio-environmental factors), the reduction in the weekly operational costs is about 7,000 €, which becomes about 37,000 € when calculating it on annual basis to be compared with the investment. Therefore, the leakage reduction could be less significant if the environmental value of water losses is not considered. However, leakages are indicators of general deterioration and pressure in the system. Therefore, the economic impact of unplanned interventions caused by the natural progress of deterioration, should be considered when performing a cost-benefit evaluation of the reduction of water losses.

Brief presentation of methodologies proposed by the participant teams

Fourteen teams from academia, research centers and companies provided their solutions for the BBLWAN at WDSA 2014. Here they are briefly presented in the order they were submitted to the conference website; thus such order does not reflect any judgment on the methodologies. Further details on the single approaches and solutions are reported in individual papers authored by each competing team.

Morley and Tricarico (2014, 2015) presented a methodology based mainly on the use of population-based optimization algorithm. They formulated the problem as a constrained single and multiple-objective optimization, implementing a generic hydraulic optimization and benchmarking software application (Acquamark – see Morley and Tricarico (2014, 2015) for details). To permit multiple solutions to be executed and evaluated in parallel a distributed computing architecture was implemented. A pressure-driven demand extension to the EPANET2 (Rossman, 2000) hydraulic model is employed to assist the optimization techniques in accurately ranking near-feasible solutions and to dynamically allocate leakage demand to the end nodes of each pipe.

Roshani and Filion (2014) presented a methodology based on a multi-objective optimization approach to minimize capital and operational costs of the network, employing NSGA-II (Deb et al., 2002). The optimization includes all the decision variables involved, e.g., pipes, valves, pumps and tanks, subject to pressure and water level in tanks constraints. The EPANET2 network solver is used to evaluate pipe leakages (simulated as pressure-dependent by means of the orifice discharge coefficient reflecting the leakage model coefficient in Eq. (3)), as well as to evaluate the hydraulic constraints (i.e., nodal pressures, tank levels, etc.). The C# programming language was used to couple the EPANET2 network solver with the NSGA-II engine. Multi-threading (parallel processing) was used to reduce the computational time.

Iglesias-Rey et al. (2014, 2015) presented a methodology combining the use of engineering judgment and an optimization model based on a pseudo-genetic algorithm. The methodology consists of two stages: an analysis of marginal costs of pipes considered for replacement, followed by the network topological analysis to study the pipes that could be potentially closed in order to facilitate pressure control. Additionally, a methodology for studying branched areas was also developed, determining possible location for pressure reducing valves. This approach was aimed at reducing the number of decision variables, thus reducing the domain of the specific optimization model in the second stage. Network hydraulic analysis has been performed using the EPANET2 network solver using emitters at nodes to simulate leakages.

Creaco et al. (2014, 2015) proposed a multi-objective optimization approach considering three objective functions (i.e., minimization of installation cost, operational cost and PRVs cost). The approach consists of four steps. First, some feasible solutions are identified based on engineering judgment. Then, for step two and three, the NSGAII optimizer was implemented to find an optimal set of solutions: firstly considering only to capital and operational costs, and then considering operational and pressure reducing valves costs. Finally, by grouping the solutions found at the end of previous optimization steps the final three-objective Pareto surface was derived and the best solution selected. The methodology implements the EPANET2 hydraulic solver simulating leakages with emitters first, and then assessing leakages using a sub-routine that applied the Germanopoulos' formula.

Price and Ostfeld (2014, 2015) proposed a methodology based on the successive Linear Programming by minimizing costs. A linear representation was solved successively for the non-linear constraints of headloss, leakage, pump energy consumption and pipe sizing. The optimization model returned minimal cost pump scheduling and pipe sizing while minimizing leakage and maintaining minimum service pressures to the consumers. The problem is divided into four main parts: PRV positioning, pumping station and water tank sizing, pipe sizing and pump scheduling for minimum leakage and operational cost. The resulting optimal pump scheduling was not controlled by the water levels in the tanks (as required by the main BBLAWN rules) as the pumps are operated to maintain minimum water pressures at the consumer nodes while utilizing minimum electrical tariff periods. For this reason the solution provided was not accepted for the competition since it was not comparable with other teams that complied with the rules.

Diao et al. (2014, 2015) proposed a methodology based on a clustering-based hierarchical decomposition. The network is decomposed into a twin-hierarchy pipeline structure consisting of backbone mains and community feeders. The method consists of three steps: clustering analysis; vulnerability analysis; and identification of backbone mains and community feeders. The system was topologically decomposed into backbone mains and 28 communities. Optimal pressure control strategies for each cluster is addressed in a sequential manner based on the cluster hierarchy with constraints on network performance. Considering such simplified topology, the most cost effective PRV placement strategy and pipe upgrading options for each branch cluster were identified.

Eck et al. (2014, 2015) proposed a methodology that decomposes the problem according to the type of intervention, considering and assessing each type in sequence. Initially, a diagnosis of the network is performed through simulating its hydraulic behavior with no infrastructure or operational modifications. An optimization technique is then developed to recommended improvements of a particular type, such as pipes to replace. The presented technique is applied sequentially to yield a list of suggested improvements for the network. The leakage simulation problem was transformed into an equivalent formulation for which EPANET can be applied. To simulate the leakage equations, an iterative technique was developed using the emitters feature in EPANET.

Tolson and Khedr (2014; Khedr and Tolson, 2015) propose to rely on engineering judgment with limited use of optimization to generate an approximation of the Pareto-optimal front without intensive computational requirements. A simple heuristic approach consisting of a five-stage approach based on enumeration and trial-and-error (WDN modeler expert judgment) was used to identify and prioritize potential decisions variables (i.e., pipe replication, PRV installation, tank installation, etc.). The decision variables are ranked based on their operational savings per unit of capital cost expenditures with those variables with the highest ratio being implemented. The system hydraulics and objective functions were recalculated after each successive change to ensure feasibility and all intermediate solutions were used to generate a trade-off curve. Finally, the quality of the Pareto-optimal curve generated using engineering judgment, was compared to one created using a heuristic global search optimization algorithm. A background leakage modelling methodology in EPANET was adopted for approximating the leak assessment methodology provided by the competition organizers.

Saldarriaga et al. (2014, 2015) presented a methodology that used the Unit Headloss to select pipes to rehabilitate, the Flow-Pressure concept to locate valves and GA for the pump optimization process. The methodology is composed of different steps, starting from the application of a leakage model to the initial network using EPANET model with emitters. The network was then sectorized according to DMA’s demand patterns and a rehabilitation process was conducted to meet pressure requirements. An infrastructure optimization process was carried on allowing for improvements, such as installation of new pipes, pumps and tanks, and a pump optimization was iteratively performed together with the estimation of leakage parameters. Finally, the whole network improvement was considered to evaluate the final cost of the proposed solution.

Matos et al. (2014, 2015) proposed an evolutionary approach that operates in an exclusively discrete solution space and is intended to require as little engineering judgment and time as possible while attaining acceptable and informative results that are useful for decision-making. Its main features are custom crossover and mutation operators, being the latter guided by specific network and simulation parameters. The developed operators, specific for water distribution network optimization tasks, are applicable to single- and multiple-objective genetic algorithms as well as to other evolutionary algorithms.

Thus, authors presented two implementations: the first consisted of a single-objective (i.e., minimization of the total operational and capital cost) genetic algorithm whose mutation operator was designed to find increasingly parsimonious solutions as the optimization unfolds. The second was a multiple-objective approach: the objectives were the minimization of investment and operational costs. A simple post-processing greedy algorithm to locally refine pipe replacements is also presented as a means of complementing the evolutionary approach.

Computations have been carried out in a Java version of EPANET aiming at increased computational efficiency, greater platform portability, and improved flexibility regarding optimization software.

Rahmani et al. (2014, 2015) presented a methodology based on a three-stage multi-objective optimization model. At the first stage, the optimal design of pipeline rehabilitation, pump scheduling and tank sizing is formulated and solved on the skeletonized network by optimizing the costs of pipes, upgrading of pumps and tank and the cost of water losses and energy. The second stage employs the best Pareto front obtained from the first stage to solve the previous two objectives optimization problem for the full network. The third step employs a three-objective optimization model by adding the number of PRVs as the third objective and PRV settings are also added to the decision variables. This stage employs three solutions on the Pareto front of the second stage to seed the optimization on the full network.

The optimization model used in all stages is non-dominated sorting genetic algorithm (NSGA-II) and the simulation model is the EPANET software tool.

Sousa et al. (2014, 2015) proposed two optimization models supported by engineering judgment to help in choosing the best strategies to follow, starting with the optimization of the pump controls, followed by the installation of PRVs and the replacement of existing pipes. The first optimization model used is a least-cost design model to identify the pipes to be replaced and size them; the second is an optimal operation model to define the pump controls and the PRV settings. Both models are solved by linking a hydraulic simulation model (WaterNetGen - a pressure driven EPANET extension) with a simulated annealing algorithm. The selection of final optimal solutions was done using engineering judgment.

Vassiljev et al. (2014, 2015) proposed an approach based on a trial-and-error methodology using heuristic methods coupled with hydraulic simulation. To find the optimal solution, customized research tools were developed for WDN optimization. These tools, based on the EPANET2 toolkit, were employed for the optimization of water tanks levels to switch pumps on/off; the estimation of the influence of PRVs on leakages to decide adding a PRV to a pipe or not; the calculation of leakages under different conditions. Commercially available tools are also used carrying out comparison of various network structures (parallel pipe alternatives). The analyses were carried out in four major stages: (a) the elimination of bottlenecks (in terms of small pipe diameter and/or low pipe roughness coefficient C); (b) the installation of PRVs to reduce the pressure at leak nodes; (c) the examination of pump efficiencies; and (d) the optimization of water levels in tanks.

Finally, Shafiee et al. (2014, 2015) implemented a genetic algorithm approach within a high-performance computing platform to select tank sizes, pump placement and operations, placement of pressure control valves, and pipe diameters for replacing pipes. Multiple problem formulations are solved that use alternative objective functions and allow varying degrees of freedom in the decision space. The original framework is based on a genetic algorithm that was written in Java and calls functions from the EPANET toolkit to simulate network hydraulics. The framework is implemented on a parallel cluster and was modified for the BBLAWN application, incorporating additional functions from the EPANET toolkit for manipulating pressure control valves and created new functions for calculating hydraulics based on leakage across pipes.

Discussion

All the approaches proposed by teams brought interesting contributions to solving the complex BBLAWN problem. The proposed strategies range from those strongly based on a multi-objective optimization including all the conflicting cost objectives and the involved decision variables (pipes, valves, pumps and tanks) proposed by the organizers (Morley and Tricarico ; Roshani and Filion), to the approaches based on successive stages in which the engineering judgment has the main role, thus resulting in a limited use of optimization procedures (Tolson and Khedr).

Most of the proposed methodologies are structured as multi-stage approaches combining it with the use of engineering judgment/expertise, which has been aimed at reducing the size of the optimization problem and driving towards the selection of intermediate and final solutions. The use of engineering judgment is very important for the extension of the proposed approaches to real-network problems, because it allows the inclusion of other types of knowledge and expertise in the technical and decision-making process.

From the optimization standpoint, most of the teams implemented population based techniques (i.e., genetic algorithms) in a multi-objective setting, including, in different combinations, the conflicting cost objectives proposed by the organizers. The only exceptions are Price and Ostfeld, who solved the problem using Linear Programming, and the approach by Sousa et al. that implemented a simulated annealing algorithm. Some other teams (Diao et al. , Saldarriaga et al. , Rahmani et al.), tried to reduce the space of solutions of the “main” multi-objective optimization by means of network clustering/sectorisation/skeletonization, thus dealing with a larger number of smaller (and simpler) optimization problems.

From the computational point of view, all teams used the EPANET hydraulic solver with some of them implemented a pressure-driven version in order to enhance the simulation of background leakages. Interestingly, Matos et al. implemented a Java version of EPANET. Some teams, (Morley and Tricarico , Roshani and Filion , Shafiee et al.) , have also made use of parallel processing in order to reduce the computational time of their applications.

As reported by many teams, the adoption of the EPANET2 model, although well-known and used worldwide, showed major limitations in dealing with the BBLAWN real problem. First, it required some modifications/post-processing of results in order to consistently assess the background leakages from pipes according to Eq. (3); otherwise the simulation is affected by errors as explained above. Second, EPANET2 does not model pressure reduction valves controlled by remote set points (i.e., far from the downstream PRV node). This limitation actually prevented all teams from using the remote control option of valve that was allowed in BBLAWN rules. However, this is a preferred option due to control solutions currently available to water utilities. Using remote controlled PRVs is likely to provide solutions that are technically more reliable than “classical” PRVs. In fact the pressure at remote set point (e.g., the critical node in the controlled area) better reflects the real network hydraulic behavior than the one immediately downstream of the PRV. For example, in case of abnormal water demand (e.g. for firefighting or during pipe burst events) resulting in pressure drop at the control node (which is usually the most critical node due to node elevation and/or building height), the PRV opens to reach the set pressure value. Vice versa, the set point of a “classical” PRV needs to be modulated over time based on some prediction of network hydraulic behavior, which relies heavily on predicted demands and model calibration (and related uncertainties).

In this regard, the solution proposed by Price and Ostfeld suggested that a more realistic problem formulation, maybe in future “Battle” editions, could also include remote control of pumps and, also, variable speed pumps.

Depending on the particular strategy adopted, the solutions presented different trade-offs between capital (parallel pumps, tank enlargement, pipe renewal/doubling) and operational (energy, water losses) costs.

Table 3 summarizes the key decision variables. The solutions showing lower capital costs, are also those requiring the highest operational costs. In fact, keeping the existing water infrastructures intact (i.e. without any investment on asset renewal) is likely to result in large volume of water losses and pumping energy requirements. On the other hand, a significant reduction in water losses can be achieved by strategically investing in renewal of pipes, enlargement of tanks and/or new pumps. Some of the solutions with the lowest capital costs are also those requiring implementation of the largest number of PRVs to control as much as possible pressure through the network. Nonetheless, the need for providing water to customers that satisfies the minimum pressure requirement, does not permit further reduction of leakages via PRVs only.

Such a variety of solutions further demonstrates the need for engineering judgment as well as the knowledge of water utilities’ management strategies to take effective and sustainable decisions in such a complex multi-objective problem encountered in a real networks.

Table 3. Comparison among different BBLAWN approaches

Conclusions

The Battle of Background Leakage Assessment for Water Networks (BBLAWN) was designed to follow the tradition of the “battle” competitions” held during the Water Distribution Systems Analysis (WDSA)

Conferences. The BBLAWN problem was about the optimal management of water and energy resources, as relevant environmental and socio-economic issue worldwide. The competition considered asset renewal planning and strengthening, as well as optimal operation, including possible installation of PRVs. All the participant teams performed well in the competition, producing interesting results and some innovative ideas worthy of future exploration. Most of the proposed methodologies were able to suggest sensible solutions in both short time (operational) and medium time (tactical) horizons.

The review of all contributions clearly shows how challenging the BBLAWN problem is from engineering perspective since it involves a real size network where multiple conflicting objectives need to be considered and realistic technical constraints accounted for. Management decisions can and should be supported by tools that combine hydraulic models capable of assessing pressure-dependent background leakages with computationally effective multi-objective optimization strategies. In order to promote the discussion inside the technical/scientific community, the rules of BBLAWN did not compel the use of any specific software for hydraulic modeling and only provided the management objectives to be fulfilled.

Due to the number of decision variables and the size of the search space, the WDN design process cannot be fully automated. Engineering judgment can and should provide invaluable support to the formal optimization approaches in the search for feasible alternative solutions. A multi-step approach was preferred by most of the teams since it permits the progressive evaluation of the improvements in WDN performance achievable at each step. The overview of proposed solutions demonstrated that many alternatives are compatible with the problem in hand, ranging from massive network renewal (at lower operational cost) to minimal interventions (requiring high cost for energy and pumping). If the same approach was adopted for real life applications, the selection of the optimal strategy and of the most effective solution, should take into account the possibility of planning different interventions over time, thus reflecting the budget available. This would make preferable, for example, in the short term horizon the optimal control of pumps rather than more expensive renewal of asset.

The overview of the proposed strategies also emphasized the need to overcome current limitations of WDN simulation models in order to permit more realistic assessment of background leakages as well as the modelling of remotely controlled devices. This would permit more reliable simulations to support WDN management, allowing also the assessment of the impact of effective ICT solution for WDN operation.

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List of Tables

Table 1. Ranking of the PRVs

Pipe ID	Pipe ID of PRV	Water Lost [m^3]	Leakage [%]	Energy [KWh]
	original	36,281	26.05	42,221
	solution 5	34,533	25.11	42,063
P122	48	32,140	23.79	41,315
P758	276	30,639	22.93	40,870
P789	299	29,458	22.24	40,365
P5	234	28,637	21.76	39,849
P305	163	27,976	21.36	39,346
P1000	441	27,395	21.01	39,379
P115	40	26,807	20.66	39,202
P1033	20	26,342	20.37	38,956
P125	51	26,049	20.19	38,898
P1002	443	25,794	20.03	38,679
P937	368	25,575	19.90	38,628
P786	296	25,240	19.69	38,548
P16	79	24,943	19.50	38,539
P772	286	24,801	19.41	38,418
P794	301	24,580	19.27	38,401
P72	267	24,370	19,.14	38,365
P344	187	24,170	19.01	38,106
P1001	442	24,075	18.95	38,004
P329	175	23,915	18.85	37,668
P1042	28	23,852	18.81	37,667
P633	255	23,823	18.79	37,695
P781	292	23,696	18.71	37,490
P1024	10	23,632	18.67	37,489
P811	316	23,583	18.63	37,474
P10	2	23,531	18.60	37,430

Table 2. Relevant data of the initial and final status of the network. Operational costs are weekly-based.

Solution	Water Loss [m^3]	Leakages [%]	Energy [KWh]	Operational cost [€]	Capital cost [€]	PRVs Cost [€]
initial	36,281	26.05	42,221	77,738	0	0
Final	23,531	18.60	37,430	51,500	62,305	26,182

Table 3. Comparison among different BBLAWN approaches

Solution by:	Hydraulic simulation and leakage model	*Complies with Eq.(3)*	Strategy approach	PRV [-]	new/ parallel pipes [-]	new/ parallel pumps [-]	Enlarged tanks [m^3]
Morley and Tricarico (2014)	A pressure-driven version of EPANET, incorporating the leakage model.	*Yes*	NSGA-II and Omni-Optimizer were used to run to completion on the full-scale multi-objective (MO) optimization.	6	373	3	-
Roshani and Filion (2014)	EPANET2; emitters at nodes to simulate leakages; orifice discharge coefficient reflecting the leakage model coefficient.	*No*	Use of NSGAII to optimize (i) the capital cost and (ii) operational costs. Decision variables identification based on technical considerations and constraints (e.g. only pipes with a diameter of 300 mm or greater are considered for possible duplication).	14	409	8	T3: +500 T4: +1000 T7: +1000
Iglesias-Rey et al. (2014)	EPANET2; emitters at nodes to simulate leakages; dummy nodes close to tanks to allow leakage calculation.	*No*	Single Objective (SO) Optimization strategy based on technical considerations (e.g. only pipes and pumps as decision variables after a cost analysis; sub-areas optimized separately; analysis of minimum pressure to change a pipe) Post-processing to improve the solution through fine adjustments based on best management practices	61	416	2	-
Creaco et al. (2014)	EPANET2; emitters at nodes to simulate leakages; second refinement to make leakage simulation compliant with the BBLAWN rules	*Yes*	Three successive optimization considering 2 objectives for each optimization; final refinement of PCV setting to reduce leakage; no parallel pipes were allowed, based on cost analysis; parallel pumps location based on engineering judgement.	44	256	-	T5: +500
Diao et al. (2014)	EPANET2; emitter nodes to simulate leakages; iterative update of nodal demand using simulated pressure.	*No*	Step-by-step optimization approach, based on hierarchical initial classification into trunk clusters and branch clusters. Next, optimal pressure control strategies for each cluster is addressed following the hierarchical sequence.	61	349	2	-
Eck et al. (2014)	Modified EPANET2 code; emitter coefficients at nodes are updated through iterations to simulate leakages.	*No*	Sequential assessment of intervention types. An optimization technique is developed and applied sequentially to yield a list of suggested improvements for the network. Optimization solvers based on Bonmin and Ipopt techniques to solve mixed integer nonlinear programming problems.	22	345	2	-
Tolson and Khedr (2014)	EPANET2; emitters at nodes to simulate leakages; dummy node added to simulate leakages close to tanks.		Bi-objective optimization: maximizes the operational savings and minimizes the total capital costs. Engineering judgment and cost analysis is heavily relied upon to identify candidate and priority decision variables (i.e., PRV valve configurations).	27	23	-	T2: +500

	No	The design is fine-tuned to ensure pressure and tank level constraints.				
Saldarriaga et al. (2014)	EPANET2; leakages along pipes are simulates with emitters at pipe downstream nodes only using an approximated emitter coefficient. *No*	The Unit Headloss concept supports rehabilitation interventions; the Flow-Pressure concept support the location of valves; GA support the pump optimization process. Sequential approach: leakage parameters estimation; sectorization; rehabilitation; PRV location; pump optimization with GA; Union of all DMAs; final leakage parameters estimation; final pump optimization; final cost evaluation.	66	12	-	-
Matos et al. (2014)	Java version of EPANET2. *No*	Preliminary engineering analysis; MO-Optimization: minimization of investment and of operational costs. A modified version of the NSGA-II was employed in order approximate the Pareto front.	12	203	2	T1: +5000 T4: +5000
Rahmani and Behzadian (2014)	EPANET2 to get approximate solutions; a posteriori application of the Geranopuolos' leakage model. *Yes*	Three-stage multi-objective optimization model (NSGAII). First stage: optimal design of pipeline rehabilitation, pump scheduling and tank sizing using a skeletonized WDS model; minimizing capital and operational costs. Second stage: the same optimization using the full network. Third stage: the same optimization including 168 hr simulation.	2	270	1	-
Sousa et al. (2014)	WaterNetGen permitting to compute burst and background leakages, consistently with BBLAWN rules. *Yes*	The methodology comprises two optimization models: a least cost design model to identify the size of pipes to be replaced and optimal pump controls and the PRV settings. A simulated annealing algorithm was used to solve the optimal WDN design and operation problem.	41	429	0	0
Vassiljev et al. (2014)	Customized tool based on EPANET2 toolkit to simulate background leakages. *Yes*	Use of customized research tools developed for tank parameter optimization (optimal volume); estimate the profit of each PRV and cost of exploitation. Four stages analysis: (i) elimination of bottlenecks, (ii) installation of PRVs, (iii) examination of pump efficiencies, (iv) optimization of tanks (pump on/off levels and tank diameter).	80	5	2	0
Shafiee et al. (2014)	Modified EPANET toolkit by creating new functions for calculating hydraulics based on background leakage across pipes. *Yes*	SO-Optimization using a GA to select tank sizes, pump placement and operations, PRV locations, and new pipe diameters. Multiple problem formulations are solved that use alternative objective functions and allow varying degrees of freedom in the decision space.	28	29	0	T4: +1000

List of Figures

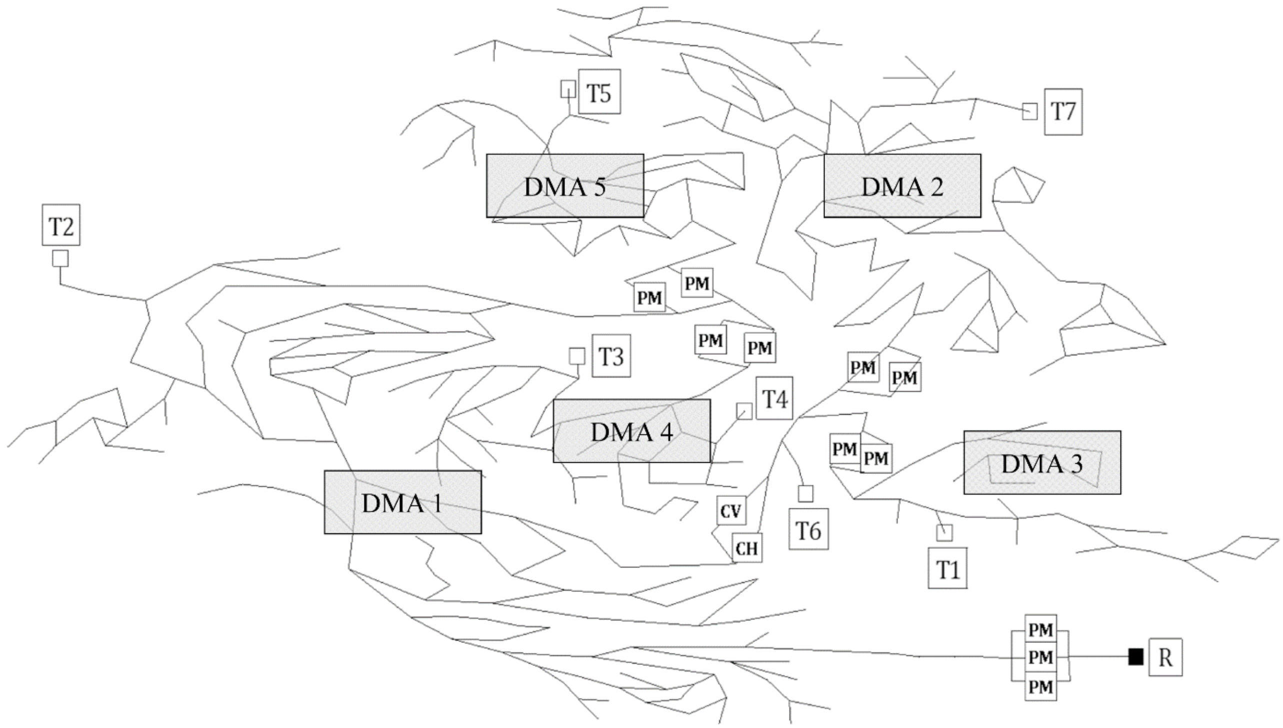

Fig. 1. TOWN-C for BBLAWN composed of 444 pipes, a reservoir (R), seven tanks (Tx), eleven pumps (PM), a control valve (CV), a check valve (CH).

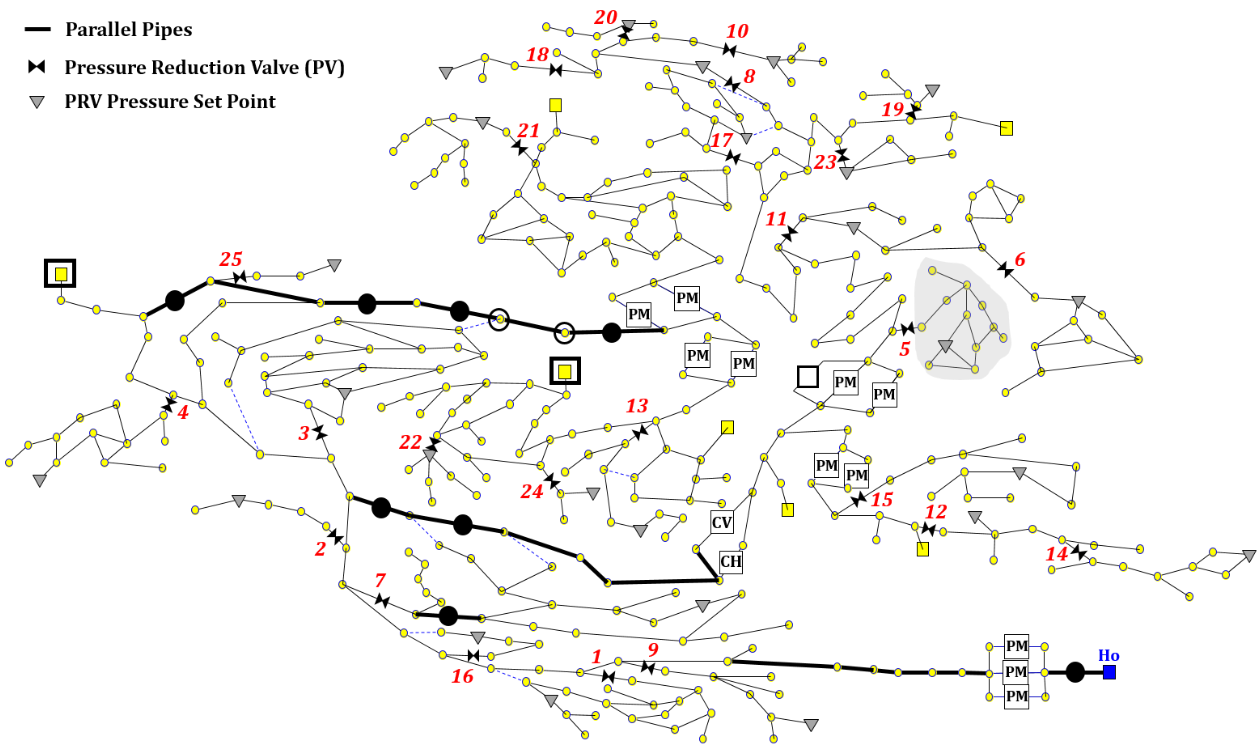

Fig. 2. TOWN-C pressure control valve (PRV) and node of pressure set (Pset).

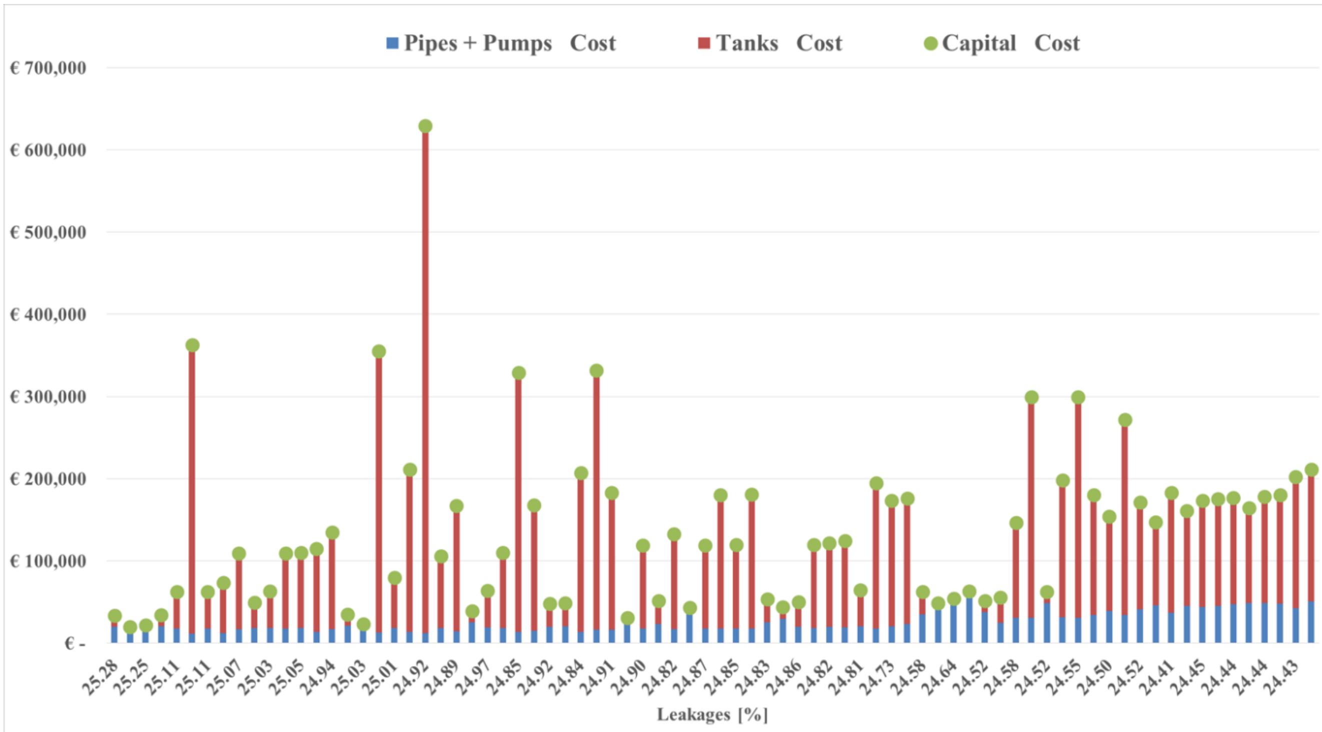

Fig. 3. Pareto front of solutions for the multi-objective optimization problem (pipe and pump cost vs. energy and water loss cost vs. tank enlarging cost).