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Battle of Background Leakage Assessment for Water Networks (BBLAWN) at WDSA Conference 2014

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Abstract

The Battle of Background Leakage Assessment for Water Networks (BBLAWN) is the fifth in a series of "Battle Competitions" dating back to the Battle of the Water Networks (BWN) in 1985 and, more recently, the Battle of the Water Sensor Networks (BWSN) in 2006; the Battle of the Water Calibration Networks (BWCN) in 2010 and the Battle of the Water Networks Design (BWN-II) in 2012. The BBLAWN asks for a design methodology for reducing water losses due to background leakages in a real water distribution system. The results have been presented at a special session of the 16th Water Distribution Systems Analysis Conference, held in Bari, Italy in July 2014, where 14 teams from academia, research centers and companies presented their solutions to the problem in hand.

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Keywords: Water Distribution Networks; Background Leakages; Asset Management; Pump Scheduling; Pressure Reduction Valves.

1. Introduction

The Battle of Background Leakage Assessment for Water Networks (BBLAWN) called for teams/individuals from academia, consulting firms, and utilities to: (i) propose a design methodology for reducing water losses due to background leakages and (ii) apply it to a real water distribution system. The results of the BBLAWN have been

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presented at a special session of the 16th Water Distribution Systems Analysis Conference, held in Bari (Italy) in July 2014 (www.water-system.org/wdsa2014).

Participants were provided with files containing instructions and supporting material and an FAQ section in the conference website. Additionally, the participants were invited to emphasize the generality of the proposed approach and the technical reasoning underlining the intended benefits of the adopted design strategy.

It should be emphasized that none of the BBLAWN organizers has taken part in the BBLAWN as participant. The organizer's responsibility was to assemble the methodological approaches and results, making sure that the BBLAWN is assessed objectively, organize the session at the WDSA2014 event, and prepare this manuscript to summarize the outcomes and results of the competition.

2. Problem description

Improvements to the water system in the municipality of C-Town [1] are being made to meet the minimum pressure for a sufficient service at 20 m of pressure and to control background leakages. The hydraulic system network layout is reported in Fig. 1.

To accomplish this task it is assumed that the city has already commissioned the development of a calibrated hydraulic model of the actual existing network to evaluate its present state and its 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 referred to tanks.

The existing infrastructure is not able to meet pressure performance targets when future demands are considered. The situation is compounded by excessive background leakage and, thus, the water utility is interested in minimizing operational and capital costs as well as background leakages. The utility is also concerned about environmental and financial damage caused by water loss as the environmental penalty for one cubic meter of water lost is fixed at 2 €.

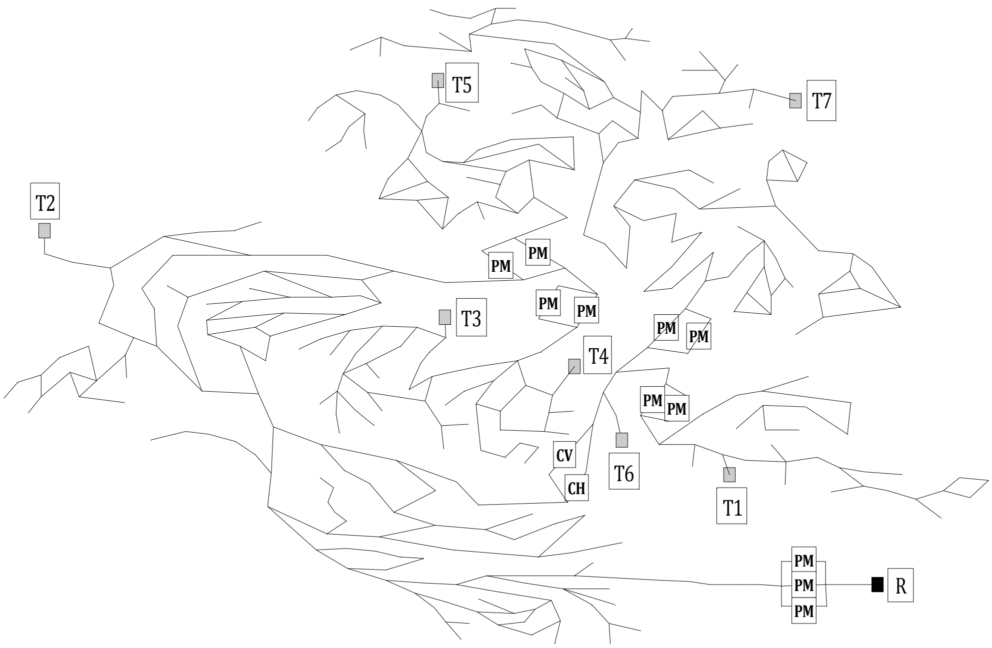

Fig. 1. TOWN-C for BBLAWN composed of 444 pipes, a reservoir (R), seven tanks (Tx), eleven pumps (PM), a control valve (CV) and a check valve (CH).

3. Design Requirements

3.1. Costs

The water utility desires a low operational and capital cost solution. In particular, the operational costs are a result of pump operations and background leakages, while capital costs are associated with the selection of the pipe and tank material and construction, and any upgrades of the existing pumping stations.

As capital and operational costs occur at different times during the lifetime of the project, annual costs are provided for the new components of the network in the section titled "Design Options". These costs take into account the lifetime of each specific component and the discount rate to be applied to the costs. Consequently, the total cost requiring minimization is the sum of the annual capital and the pumping operations and water loss costs. To account for the latter costs, the weekly cost associated with the pump power and the volume of background leakages have to be computed and multiplied by the number of weeks in a year (52). It is worth noting that, since this is a onetime intervention, no time horizon or network lifespan is assigned. In addition, capital and operational costs are provided on annual basis.

3.2. System performance under normal operation

The water utility requires that each demand node of the network has water delivered to it with adequate pressure. Nodes without demand have only the requirement of a minimum pressure being above zero. The minimum required pressure for demand nodes is 20 m. An additional requirement of the water utility is that at the end of the extended period simulation (1 week) each tank has to have at least the same volume of water it had at the beginning of the simulation (note that this initial volume has to be set equal to half the volume of the tank). Moreover, during normal operation, tanks are not allowed to empty.

Table 1. Table of pipe cost.

Pipe costs			Parallel pipes cost		
Diameter [m]	H-W coeff.	Cost [€/yr/m]	Diameter [m]	H-W coeff.	Cost [€/yr/m]
0.102	120	9.97	0.102	120	8.31
0.152	120	12.1	0.152	120	10.1
0.203	120	14.49	0.203	120	12.1
0.254	120	15.55	0.254	120	12.96
0.305	120	18.28	0.305	120	15.22
0.356	120	19.94	0.356	120	16.62
0.406	120	23.26	0.406	120	19.41
0.457	120	26.65	0.457	120	22.2
0.508	120	29.58	0.508	120	24.66
0.61	120	42.8	0.61	120	35.69
0.711	120	48.12	0.711	120	40.08
0.762	120	51.11	0.762	120	42.6

4. Design Options

4.1. Pipes

Pipe diameter options and costs for the network expansion are given in Table 1. The costs shown are inclusive of pipe construction, transport and installation (see left side of Table 1). According to the water utility, pipes can also be placed in parallel to existing pipes. However, as this implies the disruption of traffic and reconstruction of pavements and roads, the cost of duplicating existing pipes is higher than for laying new pipes, as showed in Table 1 (right). The Hazen-Williams coefficient for each new pipe is equal to 120. Parallel pipes are assumed to have the same length as the original ones.

The background leakages model for pipes is [2][3]:

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

where:

k = subscript of the kth pipe;
$P_{k,mean}$ = model mean pressure along the kth pipe in [m];
d_k^{leaks} = background leakages outflow along the kth pipe in [m^3/sec];
α_k and β_k = model parameters set as in Table 2, in [-] and [$m^{2-\alpha}$/sec];
L_k = length of the kth pipe, in [m].

The hydraulic system modelling allows lumping pipe outflows in Eq. (1) at the ending nodes using Eq. (2), where $\mathbf{A}_{np}$ is the network incidence matrix [3]. For the new pipes (laid in parallel or as expansion), β_k is 20% of those reported in Table 2.

$$\mathbf{d}_n^{leaks}=\frac{1}{2}\left|\mathbf{A}_{np}\right|\mathbf{d}_p^{leaks}=\frac{1}{2}\left|\mathbf{A}_{np}\right|\begin{bmatrix}d_1^{leaks}\\ \dots\\ d_k^{leaks}\\ \dots\\ d_{n_p}^{leaks}\end{bmatrix} \tag{2}$$

Table 2. Background leakage model parameters α [-] and β [$m^{2-\alpha}$/sec].

DMA	α	β
1	0.9	4.00E-08
2	0.9	2.00E-08
3	0.9	1.00E-08
4	0.9	2.00E-08
5	0.9	1.00E-08

Eqs. (1) and (2) apply also to pipes connected to water tanks and the mass balance accounting for leakages must be computed also at tank nodes.

4.2. Modelling Leakages

Water leakages are caused by small or large breaks in the pipes, which occur at main pipe level and along the pipe connections to properties. The technical literature classifies leakages in background and bursts (unreported or reported) depending on the level of outflow. Therefore, background leakages are diffuse and small outflows along pipes (mains and connections) depending on the deterioration status [3], i.e. on β in the model (1), which continuously run over time and causes significant water losses.

The bursts are the natural evolution of background leakages due to external forces/factors, which act on worn pipes. The model in Eq. (1) is aimed at predicting the outflows of the diffuse leakages, considering also the small unreported bursts. In fact, hydraulic modelling considering leakages is here aimed at planning the reduction of water losses thought pipe replacements, pressure reduction and optimal pump scheduling. The purpose of background leakage modelling by Eq. (1) is therefore different from modelling the single relevant burst for operational purpose as for example its detection and/or pre-localization.

To this aim, the model in Eq. (1) is dependent on the average pressure of pipes, because leakages along mains and pipe connections share the common characteristics of being dependent on pressure. Consequently, it is arguable that the average local pressure is a good technical attribute to correlate with the total outflows. In fact, the model in Eq. (1) states that the overall leakage outflow, then 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 stiffness of the asset [3].

From the hydraulic modelling standpoint, 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.

In fact, 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 kth pipe is concentrated at two water withdrawal points at the end nodes using the coefficient ½

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

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 [4] or [5].

If it is adopted, the strategy of using concentrated outflows into the ending nodes characterized by the coefficient $\beta_k L/2$, i.e. it is assumed a burst model surrogating the background leakage model, and the outflow is computed as follows:

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

This wrong assumption generates a modelling error that is unpredictable being the difference between d_i^{leaks} of Eq. (5) and Eq. (4) a function of asset and hydraulic parameters,

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

It is worth to note that if $\alpha=1$ is used, the resulting nodal outflows would be different as follows:

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

Eq. (7) shows that although Eq. (6) states that the outflow seems to be unchanged using α=1, a different distribution of pressures through the network occurs which changes the background leakage outflows. In other words, for any $\alpha\neq1$ the background leakages prediction considering the single pipe is already different, while for α=1 the prediction becomes different because of the dissimilar demand and pressure distribution in the network.

4.3. Tanks

Because of the increased demands, the water utility is also allowing for the addition of new tanks, but only adjacent to the existing tanks, where it is assumed that the water utility already owns sufficient land. New tanks are assumed to have the same height and bottom elevation as the existing adjacent tanks (because the water utility does not want to introduce new valves to control the system). All new tanks are cylindrical and come in pre-specified standard sizes shown in Table 3, together with associated annualized costs. The construction of non-standard tanks is not considered by the water utility because they are regarded as being too expensive.

Table 3. Tank annual costs.

Volume [m^3]	Cost [€/yr]
500	14020
1000	30640
2000	61210
3750	87460
5000	122420
10000	174930

Note that the annual costs shown in Table 3 already include the connectivity costs to link the new tanks to the network. Therefore, the addition of new tanks can be modelled simply by increasing the tank diameters so that the resulting volume is equal to the existing tank volume plus the new tank volume. It is not possible to cancel an existing water tank and position a new tank at a lower elevation at the same location.

4.4. Pumps

Existing pump systems can be upgraded by adding new pumps to the pumping stations in parallel to the existing pumps; there is no limitations on the maximum number of pumps at each pump station. No additional pumping stations or boosters can be placed into the network because the water utility does not have any available location for these new components. The maximum pump efficiency is 70% for existing pumps, while new pumps have a maximum efficiency of 80%. The pump efficiency curve is given as:

$$\begin{aligned} & H = H^s - rQ^c \\ & \eta = -\frac{4\eta_{max}}{Q_{max}^2} Q^2 + \frac{4\eta_{max}}{Q_{max}} Q \\ & Q_{max} = \left(\frac{H^s}{r} \right)^{\frac{1}{c}} \end{aligned} \tag{8}$$

where η_{max} = maximum pump efficiency; H^s, r and c = parameters of the pumps. Eq. (8) represents a parabolic function with the maximum value (η_{max}) at $Q_{max}/2$.

Table 4. Pump annual costs.

Ho_k [m]	r_k [$f(c_k)$]	c_k [-]	h_k [-]	C [€/yr]
70	929	1.36	0.8	4133
90	37512	2.15	0.8	3563
120	86972	2.59	0.8	4339
90	187486	2.41	0.8	3225

In order to do not increase the complexity of the problem, variable speed pumps are not allowed.

4.5. Valves

The water utility is considering introducing pressure control valves (PRVs) in the system. The target is to maintain the cost low while reducing the background leakages. The PRV costs are reported in Table 5 and it is assumed that they have the diameter nearest to the corresponding pipe. The pressure set point in controlled nodes can change over time.

Table 5. Pressure Control Valve annual costs.

Diameter [m]	Cost [€/yr]
0.102	323
0.152	529
0.203	779
0.254	1113
0.305	1892
0.356	2282
0.406	4063
0.457	4452
0.508	4564
0.61	5287
0.711	6122
0.762	6790

In addition, there are no restrictions on using pressure control valves controlled by a remote pressure set point (P_{set}). For this reason it is required to define the control node (i.e. where P_{set} should be reached), the pipe where the valve should be installed and its direction (as reported in the supporting file of results provided to participants). Pressure Reduction Valves (PRV) are assumed to work (i.e. closing) when the pressure is higher than P_{set}. Pressure Sustain Valves (PSV) are assumed to work (i.e. closing) when nodal pressure is lower than P_{set}. EPANET's Pressure Breaker Valves are not real devices and should not be considered in the design.

It is also possible to close one or more pipes at no cost since an isolation valve is assumed already present on each pipe. Thus, closing a pipe (isolation valve) to push all the water through a control valve into a DMA can be also part of the solution. However, real-time control or time scheduling cannot be defined to open/close pipes, i.e., pipes are kept closed throughout the entire extended period simulation.

Throttle Control Valve (TCV) controlled by PLC (e.g. by flow or time) are not allowed since they would require real time prediction of demand which was not commissioned by the C-Town municipality. Moreover, controlling the TCV by time is assumed to lack necessary robustness caused by the uncertainty on boundary conditions.

5. Pump controls

The pumps are controlled by water levels in tanks as reported in the EPANET input and MS-Excel files since using the hydraulic network state is assumed robust in face of uncertainty on boundary conditions. New pumps installed in parallel to existing ones are controlled by the same tank, so they switch on/off together. The controls need to be designed also for existing pumps. It is not possible to install time-controlled pumps. Pump controller by PLC are not allowed since they would require real time prediction of demand, which was not commissioned by the C-Town municipality.

6. Time step

The municipality requested the use of continuous "hydraulic" control of any device in the system. However, relevant parameters of the hydraulic simulation and device control (e.g. pumps and valves) could be provided at one hour time step because that is assumed meaningful enough considering the uncertainty, e.g. of demands, and the steady-state hypothesis behind the hydraulic modeling.

7. Electricity tariff

The electricity tariff is shown in Table 6 where the energy prices are shown in cents/kWh. For example, the price applied on Monday is 6.72 cents/kWh from 6:00 am to 7:00 am, while it is 10.94 cents/kWh from 7:00 am to 8:00 am.

Table 6. Electricity tariff.

Tariff data (€/kWh)							
Hour	Mon	Tue	Wen	Thu	Fri	Sat	Sun
0	0.0672	0.0672	0.0672	0.0672	0.0672	0.0672	0.0672
1	0.0672	0.0672	0.0672	0.0672	0.0672	0.0672	0.0672
2	0.0672	0.0672	0.0672	0.0672	0.0672	0.0672	0.0672
3	0.0672	0.0672	0.0672	0.0672	0.0672	0.0672	0.0672
4	0.0672	0.0672	0.0672	0.0672	0.0672	0.0672	0.0672
5	0.0672	0.0672	0.0672	0.0672	0.0672	0.0672	0.0672
6	0.0672	0.0672	0.0672	0.0672	0.0672	0.0672	0.0672
7	0.1094	0.1094	0.1094	0.1094	0.1094	0.1094	0.0672
8	0.1094	0.1094	0.1094	0.1094	0.1094	0.1094	0.0672
9	0.1094	0.1094	0.1094	0.1094	0.1094	0.1094	0.0672
10	0.2768	0.2768	0.2768	0.2768	0.2768	0.1094	0.0672
11	0.2768	0.2768	0.2768	0.2768	0.2768	0.1094	0.0672
12	0.2768	0.2768	0.2768	0.2768	0.2768	0.1094	0.0672
13	0.2768	0.2768	0.2768	0.2768	0.2768	0.1094	0.0672
14	0.2768	0.2768	0.2768	0.2768	0.2768	0.1094	0.0672
15	0.2768	0.2768	0.2768	0.2768	0.2768	0.1094	0.0672
16	0.2768	0.2768	0.2768	0.2768	0.2768	0.1094	0.0672
17	0.1094	0.1094	0.1094	0.1094	0.1094	0.1094	0.1094
18	0.1094	0.1094	0.1094	0.1094	0.1094	0.1094	0.1094
19	0.1094	0.1094	0.1094	0.1094	0.1094	0.1094	0.1094
20	0.1094	0.1094	0.1094	0.1094	0.1094	0.0672	0.1094
21	0.0672	0.0672	0.0672	0.0672	0.0672	0.0672	0.0672
22	0.0672	0.0672	0.0672	0.0672	0.0672	0.0672	0.0672
23	0.0672	0.0672	0.0672	0.0672	0.0672	0.0672	0.0672

8. Design evaluation

Participants have submitted the adopted methodology and one or more solutions, but only one of them has been evaluated.

The solutions were ranked based on:

- Pipe, Pump and Tank upgrading costs.
- Water loss and Energy costs.
- Pressure Control Valve cost.

In addition, the number of people involved (i.e., in each group) has been considered, and the groups were ranked accordingly, as a surrogate measure of the consultancy cost. Consistently, a lower cumulative age was the secondary ranking criteria, because it was assumed that younger consultants are cheaper, but have limited expertise.

Finally, during the presentation of the works in Bari, the groups were ranked by a jury/committee that has taken into account the methodology and survey results from the attendees at the special session.

9. Final Remarks

The main aim of the Battle was to pose a relevant technical problem of preserving resources through the optimal asset management and assess the technical readiness of various teams. To this purpose 14 teams from academia, research centers and companies presented their solutions to the problem in hand during the conference WDSA 2104 held in Bari (Italy). Apart from the fun of competing in the Battle, the key idea is to address the need for reducing background leakages not only because of the operational costs (chlorination, pumping energy, etc.) but also for environmental sustainability purposes. Background leakages, being dependent on average pipe pressures, are a good indicator of the asset condition.

Finally, optimal pump scheduling is a problem of energy reduction not only related to tariffs but also connected to carbon footprint reduction. All support material and data were available in [6].

References

[1] A. Ostfeld, E. Salomons, L. Ormsbee, J.G. Uber, C.M. Bros, P. Kalungi, R. Burd, B. Zazula-Coetzee, T. Belrain, D. Kang, K. Lansey, H. Shen, E. Mcbean, Z.Y. Wu, T. Walski, S. Alvisi, M. Franchini, J.P. Johnson, S.R. Ghimire, B.D. Barkdoll, T. Koppel, A.Vassiljev, J.H. Kim, G. Chung, D.G. Yoo, K. Diao, Y. Zhou, J. Li, Z. Liu, K. Chang, J. Gao, S. Qu, Y. Yuan, T.D. Prasad, D. Laucelli, L.S. Vamvakeridou Lyroudia, Z. Kapelan, D. Savic, L. Berardi, G. Barbaro, O. Giustolisi, M. Asadzadeh, B.A. Tolson, R. Mckillop, The Battle of the Water Calibration Networks (BWCN). J. Water Res. Plan. and Manage., 138 (2012) 523–532.

[2] G. Germanopoulos A technical note on the inclusion of pressure dependent demand and leakage terms in water supply network models, Civil Eng. Syst., 2 (1985) 171–179.

[3] O. Giustolisi, D.A. Savic, Z. Kapelan, Pressure-Driven Demand and Leakage Simulation for Water Distribution Networks. J. of Hydraulic Engineering, 134 (2008) 626 – 635.

[4] O. Giustolisi, E. Todini, Pipe hydraulic resistance correction in WDN analysis. Special Issue on WDS Model Calibration, Urban Water Journal, 6 (2009) 39 – 52.

[5] O. Giustolisi, Considering actual pipe connections in WDN analysis. J. of Hydraulic Engineering, 136 (2010) 889 – 900.

[6] Support material and data of BBLAWN. Available at: www.water-system.org/wdsa2014/index155a.html?q=content/battle-water-networks or www.hydroinformatics.it/BBLAWN-wdsa2014.