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Hydraulic system modelling: background leakage model calibration in Oppegård municipality

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Abstract

Advanced hydraulic models of water distribution network (WDN) permit to simulate pressure-dependent background leakages from pipes, thus being of direct relevance to support management and planning aimed at water losses reduction. Nonetheless, the "accurate" simulation of real WDN functioning requires the calibration of the leakage model parameters in addition to pipe hydraulic resistances. This work reports a procedure for the calibration of background leakage model and its application to Oppegård WDN. The possibility of combining prior technical insight with field observations permits to calibrate a phenomenological model which is effective for supporting technical decisions, besides limitation of the available information.

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1. Introduction

Hydraulic models of water distribution networks (WDNs) are of primary importance to support their analysis, planning and management providing technicians with information about current and/or expected system functioning consequent to planned activities. WDN hydraulic modelling is based on the fulfilment of mass and energy balance equations at pipes (links) and nodes (junctions) that underlie the physical behavior of the system [1, 2]. The strategic importance of WDN models has motivated the continuous development of algorithms for solving the

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abovementioned non–linear system of equations and software applications aimed at supporting various technical tasks.

Actually, most of commercial and non-commercial software applications were primarily developed to simulate WDN hydraulic functioning for the purpose of designing new systems, thus entailing a "classical" modeling approach under key simplified assumptions. For instance, most of these modeling tools assumed fixed demands at node allowing the so-called "demand-driven analysis" (DDA) of the WDN that permitted to verify whether sufficient pressure was reached under given network configuration (e.g. design solutions). Consistently with such perspective, most of these modeling tools were based on the assumption of invariant WDN topology, while the closure of some pipes (e.g. pump shutdown, valve closure) was simulated by using numerical expedients in order to permits the solution of the mathematical system of equations. In addition, since most of the original codes date back to the eighties, they do not permit to simulate recent advanced control devices based on current ICT capabilities like, for example, the real time control of pressure regulating valves based on the transmission of pressure measurement values from remote "critical" WDN nodes. The non-commercial software application EPANET2 [3] is probably the best-known and used WDN modelling tool worldwide, which entails such "classical" modelling approach.

Besides the pervasive use of these tools among technicians, their application for the management and operation of existing (aged) WDNs revealed that the above modelling simplifications represent actually major limitations to conduct reliable system analyses [4]. In fact, existing water distribution infrastructures are characterized by asset deterioration which results, besides other effects, into progressively increasing water leakages from pipelines. These, in turn, represent waste of water and energy resources, accelerate asset deterioration, increase non-revenue for the water utilities and, from hydraulic perspective, increases head losses up to possible pressure deficient conditions in some areas. Nowadays, the impact of water losses in WDNs is a major issue in many European systems, which were built in the last century and are now experiencing the effects of ageing.

The technical-scientific research has produced some key progresses in this direction. Based on "pressure-driven analysis" (PDA) paradigm [4, 2] advanced WDN models permit nowadays to assess background leakages from pipes (e.g. [5]), while allowing the simulation of remotely controlled pressure regulation devices. Nonetheless, from WDN modelling perspective, background leakage models introduce additional parameters to be estimated, beyond pipe hydraulic resistances which normally need to be calibrated in "classic" WDN models in order to get "accurate" simulation of WDN functioning by maximizing the matching between simulated and measured flow/pressure data.

The problem of WDN model calibration was faced by many authors primarily considering pipe hydraulic resistance value (e.g. as reported in [6]). Among the different approach proposed, Walski [7] first reported that, in order to avoid a mere error compensation procedure and achieve a robust model, the calibration should be performed considering field observations corresponding to more than one system hydraulic status while knowing pump pressures, tank elevations, and valve settings at each reported time step. Also, Todini [8] emphasized that a unique set of steady-state data is not sufficient to guarantee the network observability for looped systems even if all the nodal heads and demands are assumed as known. Thus, the use of several independent sets of steady-state observations of the hydraulic system (as for example the use of the extended period simulation - EPS) is mandatory. Greco et al. [9] further emphasized that WDN model calibration is affected by many sources of uncertainties and assuming some prior values of pipe roughness coefficients based on engineering knowledge of the WDN can be beneficial in finding robust calibration solutions. Accordingly, the objective function was formulated as a sum of squared differences between model predicted and the *a priori* estimated pipe friction coefficients.

Differently from "classical" modeling paradigm, the advanced models requires the calibration of hydraulic pipe resistances and leakage model parameters [5, 2] and software packages aimed at supporting WDN model calibration should have as candidate decision variables both types of parameters, being able to incorporate information coming for field pressure/flow readings and prior assumption based on engineering criteria.

It is worth to observe that, in some real contexts the pressure regime through the WDN is almost invariant over typical operating cycle because they are actually oversized with respect to normal water requests and/or the terrain morphology results into high pressure regimes. This is the case of Oppegård municipality (Norway), where normal pressure values can exceed 70÷80 m in large areas and the advanced hydraulic modelling is the basis for planning remote smart pressure regulation using remote real time control (RRTC) strategies for reducing water losses. In this case, a *phenomenological* WDN model is needed for predicting leakage reduction achievable by implementing different possible RRTC configurations. In addition, in such circumstances, the calibration of the leakage model

parameters is much more effective that the calibration of pipe hydraulic resistances, whose variation among technically consistent range does not produce any sensible reduction on pressure and, in turn, on water leakages.

The remainder of the paper briefly summarized the leakage modeling paradigm assumed in this work as well as the calibration strategy implemented in the WDNetXL system [10], which was used as the main platform for analyzing Oppegård WDN. Thereafter Oppegård WDN is introduced and calibration results are reported and discussed.

2. Representing background water leakages in WDN models

Real water losses in WDNs can be conceptually classified into *burst* and *background leakages* [11]. *Bursts* are intended as major water outflow events that are usually reported to water utilities and repaired since they are likely to produce major service disruptions. For this reason *burst* are commonly considered as accidents whose impact on WDN can be limited by improving active leakage control and the efficiency of detection and repair actions.

Vice versa, *background leakages* are intended as outflows running from small cracks, holes, deteriorated joints or fittings, occurring along pipes. As diffused water outflows, *background leakages* do not result into evident and quick pressure drops through the network, thus they are not reported and run for longer time, producing relevant impact in terms of WDN water lost volumes. For this reason background leakages can be reduced by planning medium-long term asset rehabilitation and pressure management. The latter option, is usually considered first since it is cheaper and easier to implement, as for the case of Oppegård WDN.

Accordingly, medium-term planning should be supported by a WDN simulation model that incorporate a pressure-dependent model for background leakages. In this work the model proposed by Germanopoulos [12] is considered where the background leakage outflow along the kth pipe (d_k^{leaks}) is a function of the average pipe pressure:

$$d_k^{leaks} = L_k \beta_k \left(P_{k,mean} \right)^{\alpha_k} = L_k \beta_k \left(\frac{P_i + P_j}{2} \right)^{\alpha_k} \tag{1}$$

Last term of Eq. (1) reports that the average pipe pressure can be computed as the mean value of pressure simulated at pipe end nodes j and i; L_k is the length of the kth pipe; β_k and α_k are the background leakage model parameters. The hydraulic simulation model in the WDNetXL system [10] implements the pressure-leakage model as in Eq. (2) [5]:

$$d_k^{leaks} = \begin{cases} \beta_k L_k P_{k,mean}^{\alpha_k} & P_{k,mean} > 0 \\ 0 & P_{k,mean} \le 0 \end{cases} \tag{2}$$

Since background leakages represents just one of the demand components in a WDN model [2], pipe outflows in Eq. (2) are then lumped at ending nodes using Eq. (3), where $\mathbf{A}_{np}$ is the network incidence matrix [5], $\mathbf{d}_n^{leaks}$ is the vector of background leakage demand lumped at nodes which are summed to other demand components (i.e. user water requests, hydrants (if any), etc.).

$$\mathbf{d}_n^{leaks} = \frac{1}{2} \left| \mathbf{A}_{np} \right| \mathbf{d}_p^{leaks} = \frac{1}{2} \left| \mathbf{A}_{np} \right| \begin{bmatrix} d_1^{leaks} \\ \text{K} \\ d_k^{leaks} \\ \text{K} \\ d_{n_p}^{leaks} \end{bmatrix} \tag{3}$$

It was recently demonstrated [13] that the representation of background leakages as reported in Eq. (1), (2) and (3) differs from concentrated outflows (e.g. hydrants) at nodes whose free discharge outflows depends on local pressure (P_i and P_j respectively), resulting also into different nodal demands and pressure distribution in the network.

β_k and α_k are model parameters that should be estimated for each pipe, although the following preliminary considerations can be drawn.

The exponent α_k actually entails the relationship between pressure and leakage outflow and was investigated by many authors (e.g. [14]). Although its meaning reflects the pressure-discharge hydraulic relationship of free orifices (i.e. based on Torricelli law), such exponent actually has a statistical meaning since it refer to the entire pipeline where both the location and shape of each single "leakage orifice" are unknown.

Previous studies (e.g. [15, 16]) reported various ranges of possible values for α_k encompassing different pipe materials, with possible variations between 0.5 and 2.0. Nonetheless, values of about 1.0÷1.2 were found to provide acceptable results for a wide range of material (including those installed in Oppegård WDN).

It is worth noting that, being α_k the exponent of pressure in WDN models like Eq. (1), small variations of α_k might result into significant changes in background leakage outflow. This fact is likely to result into difficulties in converging towards stable calibration results and would require to bound the numerical values of α_k in a narrow range.

Based on such considerations, the exponent α_k can be assumed *a priori* based on pipe material (i.e. by adopting values suggested from literature) and, considering possible uncertainties surrounding local outflow conditions, it can be realistically assumed to be the same over the entire network.

From hydraulic standpoint the parameter β_k, as reported in Eq. (1), can be considered as the background leakage outflow running from a unit length pipe under unit mean pressure ($P_{k,mean}$). Accordingly β_k entails the propensity to leak of the single pipe due to deterioration of water mains, customer service pipes and various connections. Water pipe deterioration is a monotone process which is statistically influenced by some primary factors like age, diameter and material [5]. In fact, pipe age is considered as the most important variable describing the increase in pipe failure rate over time due to deterioration of joints and pipes. Moreover, small diameter pipes show higher failure rate than larger ones, probably due to poor quality of workmanship involved in laying down and maintaining these pipes. Age, material and diameter are also usually the pipe information available to large number of municipalities and water companies.

Since β_k is likely to encompass all pipe features that represent the likelihood of failure, its range of variation is wider than α_k and depends on both pipe conditions and pressure regime. For this reason β_k needs to be calibrated minimizing the mismatch between model prediction and field observations.

Similarly to unit pipe hydraulic resistance [6] the background leakage model coefficient β_k also encompasses all uncertainties about, for instance, the actual pipe deterioration conditions, the characteristics of surrounding soil, the fatigue effects due to possible pressure oscillation (e.g. due to transients that are neglected in steady-state hydraulic simulation). Since the estimate of such parameter by direct pipe inspection is technically not feasible, all these considerations suggest that a credible calibration of parameters β_k should reflect similar propensity to leak of groups (i.e. cohorts) of pipes sharing similar pipe features.

3. Leakage model calibration strategy

The WDN model calibration problem consists in estimating numerical parameters that, when input into a hydraulic simulation model, yield a reasonable match between measured and predicted pressures and flows in the network, which is usually referred to as "inverse" problem [8]. Following the remarks raised in the previous section, the only parameters to be actually calibrated in leakage model in Eqs. (1) and (2) are the coefficients β_k, while exponent α_k can be assumed *a priori*. In addition, the problem of calibrating β_k share the same mathematical framework as previously reported for the calibration of unit hydraulic resistances and poses similar technical considerations [6].

As first the inverse problem is still non-linear due to the non-linearity of the system of equations underlying the WDN hydraulic modelling. In addition, in real systems, flow/pressure measurements are usually available in a limited number of pipes/nodes, thus making the problem not-determined. A technically consistent countermeasure

for this is represented by grouping pipes based on similar propensity to leak as described by pipe asset features like age, diameter and material (see previous section).

Prior values of similar β_k can be estimated based on information available at the water utility like, for example, global water balance or preliminary estimation of leakages in some WDN areas based on system knowledge and historical flow measurements in district metering areas.

The uncertainty surrounding actual customers' water demands and possible noise in flow/pressure measurement is likely to affect also the estimation of β_k. For this reason, similarly to the calibration of hydraulic resistances [7], a unique set of steady-state data would result into mere error-compensation providing non-credible results over different operating conditions. Thus, also for calibrating β_k the use of several independent steady-state sets of observations of the hydraulic system as well as the EPS of the network is mandatory [8]. As for hydraulic pipe resistances, minimizing the distance between parameters estimated during the calibration process and the prior values, permits to restrain the effect of the uncertainty in WDN model calibration [9].

The main difference with respect to the calibration of "classical" hydraulic models stems from the need to perform a PDA of the network since leakages are pressure-dependent demand components.

The WDNetXL system currently includes a function to support WDN model calibration encompassing both pipe unitary hydraulic resistances (i.e. K_k) and leakage model parameters β_k in a single calibration run. The function also permits to define two different grouping of pipes into cohorts sharing similar values of K_k and β_k, respectively.

The problem of WDN calibration is formulated as the following multi-objective optimization problem:

$$
\begin{aligned}
& f_1 = \min_{\beta_k, K_k} \left\{ F\left(\left| \mathbf{Q}_{p_s,t} - \mathbf{Q}^{obs}_{p_s,t} \right|, \left| \mathbf{H}_{n_s,t} - \mathbf{H}^{obs}_{n_s,t} \right| \right) \right\} \\
& f_2 = \frac{1}{N_\beta} \sum_{k=1}^{N_\beta} \left| \phi_k - 1 \right| \qquad \text{with } \phi_k = \frac{\beta_k}{\beta_k^{prior}}, \ \ \varphi_k = \frac{K_k}{K_k^{prior}} \\
& f_3 = \frac{1}{N_K} \sum_{k=1}^{N_K} \left| \varphi_k - 1 \right| \\
& \text{subject to:} \qquad (4) \\
& \left[\begin{array}{lcl} \mathbf{A}_{pp}\left(K_k\right)\mathbf{Q}_p\left(t\right) + \mathbf{A}_{pn}\mathbf{H}_n\left(t\right) & = & -\mathbf{A}_{p0}\mathbf{H}_0\left(t\right) + \mathbf{H}_p^{pump}\left(t\right) \\ \mathbf{A}_{np}\mathbf{Q}_p\left(t\right) - \left[\mathbf{d}_n^{leaks}\left(\mathbf{H}_n, \beta_k, t\right) + \mathbf{d}_n\left(\mathbf{H}_n, t\right) \right] & = & \mathbf{0}_n \end{array} \right] \ \left[\text{PDA model (EPS)}\right]
\end{aligned}
$$

Where F is a function of the distance between measured (*obs*) and simulated flows (**Q**) and heads (**H**) at p_s pipes with flow measurement and n_s nodes with pressure measurements, if any. The function F extends over multiple time steps t where simulated values are obtained by EPS of pressure-driven analysis (PDA) [5] reported in square brackets, running considering candidate variables K_k and β_k. $\mathbf{A}_{np} = \mathbf{A}^{\mathrm{T}}_{pn}$ and $\mathbf{A}_{p0}$ are network incidence sub-matrices, $\mathbf{A}_{pp}(K_k)\mathbf{Q}_p(t)$ is the column vector of pipe head losses at time step t containing the terms related to internal head losses of pump systems, minor head losses and evenly distributed head losses, thus the diagonal matrix depends on unit hydraulic resistance values K_k. $\mathbf{H}_p^{pump}$ is the column vector of static heads of pump systems installed along pipes (if any) varying over time t in variable speed pump cases or when pumps are controlled; $\mathbf{d}_n$ is the vector of all demand components [2] lumped at nodes (e.g. users' water demand, discharges from hydrants, filling of variable level tanks, etc.), except for background leakages that are explicitly reported in vector $\mathbf{d}_n^{leaks}$ (computed as in Eq. (3)). Both $\mathbf{d}_n$ and $\mathbf{d}_n^{leaks}$ depends on pressure (head) simulated at time step t.

Functions f_1 and f_2 represent the distance of unit pipe hydraulic resistance and leakage model parameters from prior values. The number of decision variables in the optimization problem in Eq. (4) equals the number of cohorts (N_β) including pipes with "homogeneous" expected values of β_k and the number of cohorts (N_K) of pipes with "homogeneous" unit hydraulic resistance K_k. Actually, in the problem formulated as in Eq. (4), the decision variables are the factors ϕ_k and φ_k that have to be multiplied by the priors in order to get the calibrated values of K_k and β_k. This strategy permits, wherever needed, to define only one pipe cohort, when prior values (even different among pipes) are believed to be sensibly accurate with respect to the modeling purposes, and a unique calibration factor

might permit small corrections without increasing the number of decision variables (this feature was used while calibrating the hydraulic model in Oppegård municipality, as reported in more details below).

4. Oppegård municipality

Oppegård municipality is located at south of Oslo and the WDN feeds about 40,000 inhabitants, including also the municipality of Ås. Like many other cities in Norway, the Oppegård territory is characterized by strong changes in elevation varying from 40 to 180 m a.s.l. In addition, the WDN is oversized with respect to normal water requests mainly due to high water flow to be guaranteed under fire-fighting scenario with about 20 m pressure at all nodes in the network. According to Oppegård municipality, the pressure range between 30 and 80 m, although in practical terms it reaches 130 m in some high-pressure zones. As a consequence, the pressure regime does not change during the day, in spite of the variation of the water requests from customers. The average level of leakages (based on 2012 water balance) is of 43 l/s. The water utility reported that, leaks result from pipe internal and/or external corrosion; the majority of leaks or breaches were observed to happen on grey cast iron pipes laid before 1970.

The Oppegård WDN was studied as part of the project *InnoWatING* - Innovation in Water Infrastructure - New Generation (Norwegian Research Council) to demonstrate the effectiveness of smart pressure control strategies to control pressure and reduce water leakages. The main aim of the WDN modelling was to support the planning of optimal pressure management strategies using Remote Real Time Control of Pressure Reduction Valves (PRV).

5. Calibration of Oppegård WDN model

For the purpose of planning remote pressure control schemes, the WDN hydraulic model is required to provide a representation of system behavior in terms of estimation of leakage reduction achievable by implementing alternative RRTC solutions. Thus, although the WDN model calibration should provide the best possible matching between observed and modelled WDN hydraulic status, in this case it will be used to compare various planning solutions rather than provide an accurate estimate of the absolute value of leakages in the WDN.

The calibration of the WDN model is reported here based on information which were already available to water utility, while permitting possible refinement of the calibration as soon as additional field data are collected (e.g. even using flow/pressure gauges that will be progressively installed according to the planned RRTC scheme).

The observation that the oscillation of customers' demand does not produce significant variation in pressure regime in Oppegård hints that, even changing pipe hydraulic resistances (K_k) within technically meaningful range, it will not result into appreciable variations of pressure at nodes, does will not be significant to estimate background leakage reduction under different RRTC alternative plans. As a consequence, the calibration of hydraulic resistances is performed considering prior values of K_k based on pipe age and diameter assuming one factor φ_k for all pipes in Eq. (4).

The prior values K_k were computed assuming Hazen-Williams coefficients for different diameters for cast iron pipes and different ages [17].

Following the discussion reported in section 3, the value of exponent α_k in Eqs. (1) and (2) was assumed *a priori* equal to 1.0 for all pipes since it is consistent with the average values reported in literature for cast iron pipes. Accordingly, parameters β_k were calibrated only by preliminary grouping similar pipes into 19 cohorts based on the following classification criteria:

- Age classes [yr]:]0,20];]20,40]:]40,60]
- Diameter classes [mm]:]0,100];]100,200]:]200, 350];]350, 500]
- Material classes: [Ductile Cast Iron]; [Grey Cast Iron]; [Unknown]

Prior values of β_k were obtained from the preliminary estimation of leakages at every node of the system that was provided by the water utility based on past water balance data and expected pressure regime. Such values were originally provided as an additional component of nodal demand, which follows an hypothetic time pattern of "leakage" outflow that was assumed to be opposite to the demand pattern reported for household consumption (i.e. minimum leakages where reported at peak household demand times and vice versa). In order to assess prior values

of β_k, the discharge coefficients of some hypothetic free orifices (with pressure exponent equal to 1.0) were computed assuming nodal pressure values obtained by the DDA. Such discharge coefficients were used to estimate the prior values of β_k for each pipe in the network. Finally, the length-weighted mean of those values are assigned as prior β_k for each homogeneous group. The comparison between the total background leakages computed by the pressure-driven model and that reported by the water utility permitted scaling of the prior values to be consistent with the observed annual water balance, which is about 3740 m^3/day.

5.1. Limitations in available data for calibration purposes

Fig. 1 reports the location of pressure and flow meters in Oppegård WDN system, where 24 hours of measurements were collected by the water utility.

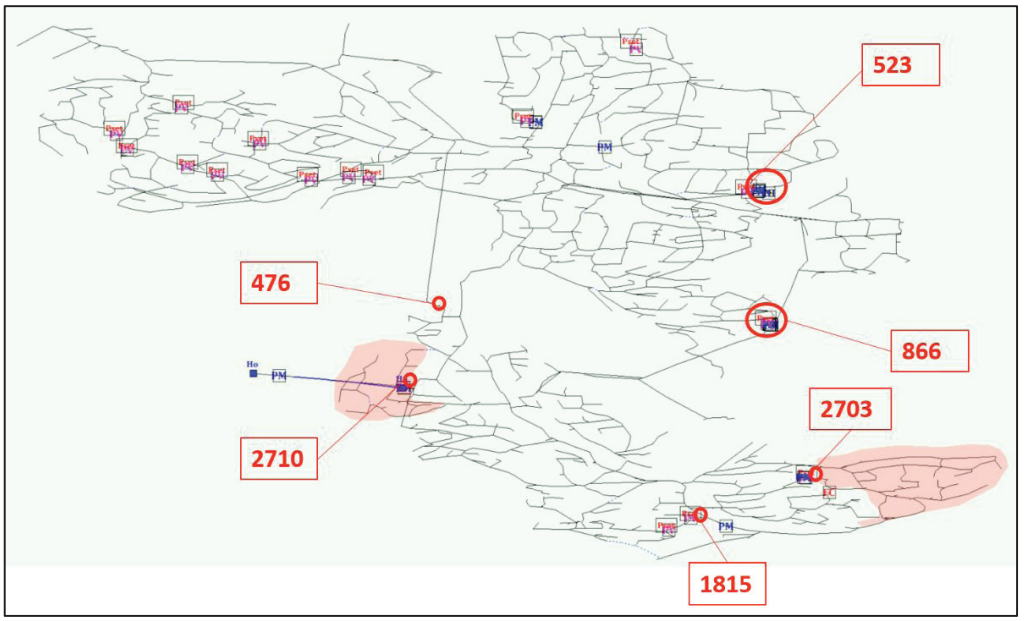


Fig. 1. Layout of Oppegård WDN and location of flow observations.

It is worth noting that, except for flow meters at pipes 476 and 1815, all observations provided refer to pumping stations. Nonetheless, the consistency of the simulation of pumps in the model with the actual functioning of the system was strongly conditioned by the lack of information about the controls for single pump switch. This is actually a significant limitation of the calibration procedure which is likely to result into abnormal simulated pressure-flow combinations reflecting the functioning of the pump. In order to mitigate the effect of such missing information on the calibration process, the pressure at pumping station outlet nodes was emulated by putting a PRV, whose set point (right downstream the PCV) is that reported in pressure records. Indeed, setting pressure at such node means to constraint the pressure downstream (i.e. through the WDN) which directly affect (pressure-dependent) background leakages simulated by the model.

Due to such limitations, all pressure measurements at pump outlet nodes were no longer included among the n_s monitored nodes; vice versa, all flow measurements were used during the calibration procedure.

It is worth noting that data were not collected during the same day and, in some cases (e.g. data on the tank close to pipe 523 in Fig. 1) they refer to Saturday which is likely to entail a different patterns of water demand with respect to a typical working day.

Finally, the demand patterns provided by the water utility where based on some "typical" demand profile, which reflect previous year water balances, but does not necessarily represents the day for which flow/pressure data were collected by the water utility for the purposes of calibration.

5.2. Oppegård WDN calibration results

The calibration procedure returned several solutions entailing different trade-offs between distances from prior values of parameters and differences between simulated and observed flow and pressure values. The selected solution was that showing the minimum average and absolute mismatching between predicted and measured data. Such solution also showed acceptable distance from priors since the average value of estimated factors ϕ_k (i.e. referred to the parameter β_k) was 1.45, with a maximum value of about 11 for one pipe cohort involving 65 pipe only.

The analysis of the differences between the model prediction and observed values revealed a twofold behavior.

For some flow observations (*i.e.* pumps at links 2710 and 2703) the differences are consistent with the uncertainties surrounding actual customers' demand and pump functioning; in addition the model over-estimates or under-estimates the observed flow values in different hours of the day, while preserving the general time pattern (*i.e.* alternative minima and peak values). Fig. 2 shows the comparison between observed and simulated water flows at pumping stations 2710 and 2703.

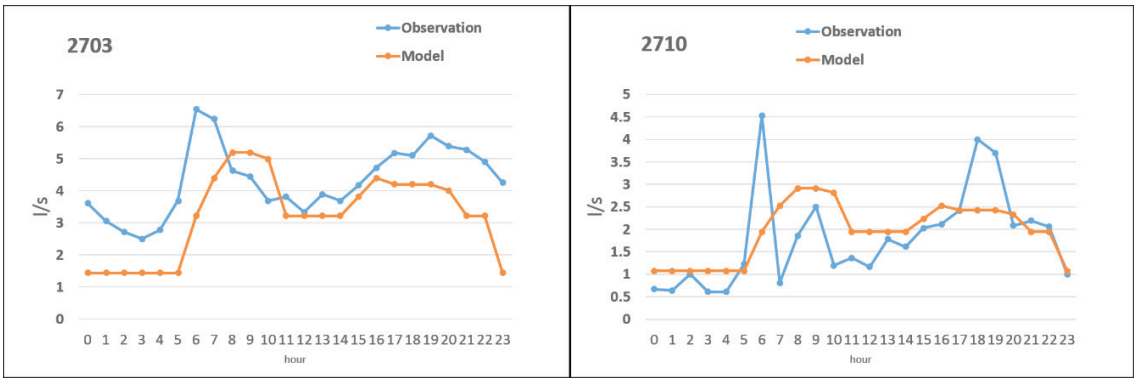


Fig. 2. Calibrated and observed flow at pumping stations outlets (pipes 2703 and 2710 in Fig.1).

For the other flow observations, the modelled time pattern is consistent with the observations, although the model overestimates the observation; Fig. 3 reports the observed and simulated water outflow from tank (i.e. pipe 523) and from the main pumping station (pipe 866), that feed about 2/3 of the WDN area. Such behavior is likely due to all limitations mentioned in the previous sub-section, including also some possible residual errors in pipe connectivity (i.e. some closed pipes not correctly reported in the model) or even the erroneous assessment of the volume of real losses (which could be much higher than that initially declared).

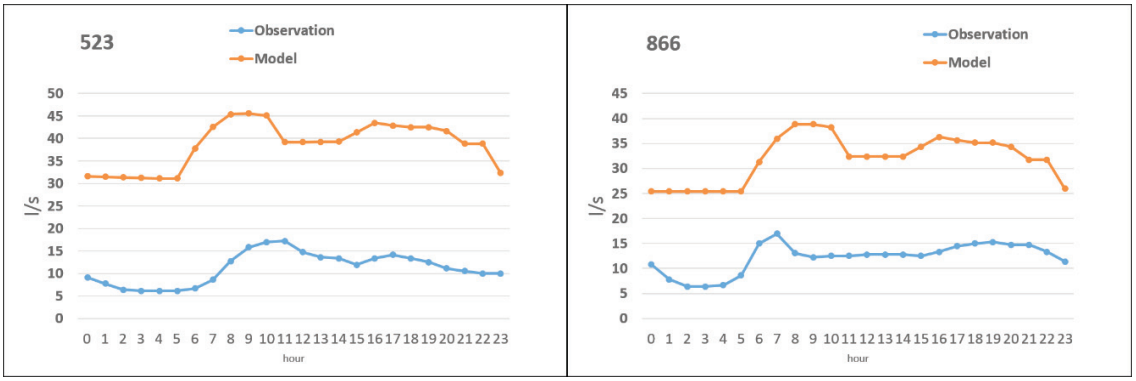


Fig. 3. Calibrated and observed flow at pumping stations outlets (pipes 523 and 866 in Fig.1).

Actually, the different behavior reported in Fig. 2 seems to indicate that water demands (mostly in terms of base values provided by the water utility through the network) at nodes are likely to have the highest influence. In fact, pumping stations 2710 and 2703 feed small areas, where the effect of demands is limited. Vice-versa, flow through other observation pipes accounts for water requests in larger areas, thus compounds the effect of mismatching between real water demand and that assumed in the model.

Due to the impossibility of correcting water demands without additional field information, the only technically feasible working hypothesis was to preserve the consistency with the average daily volume of real losses (i.e. 3740 m^3) which was reported by the water utility. In fact, since pressures in the network are fixed at values close to the observed ones (*i.e.* by using PRV downstream pumping station) such a hypothesis keeps the deterioration parameters β_k at values that are consistent with the total volume of lost water.

This assumption permits, in turns, to achieve a *phenomenological* model that can still be used to assess the effectiveness of pressure control in terms of reduction of background leakage volume achievable by reducing pressure in some zones according to consistent values of the parameters β_k which represent pipeline deterioration (i.e. propensity to leak). The final values of β_k range between about 4.7×10^{-10} to about 2.8×10^{-8}.

6. Conclusions

Managing water losses is a major issue for water utilities worldwide, which requires consistent and realistic hydraulic simulation of the systems aimed at supporting decisions on WDN operations (e.g. pressure control) and planning (e.g. asset rehabilitation). Accordingly, advanced hydraulic models should be able to simulate pressure-dependent leakages along pipes (i.e. background leakages) in order to permit technicians to evaluate different alternative strategies for intervention. Nonetheless, this poses the modelling problem of calibrating additional numerical parameters which are required in the leakage model, beyond the calibration of pipe hydraulic resistances.

This contribution first reports a WDN model calibration strategy which permits to simultaneously assess the unitary hydraulic resistances of pipes and the parameters of background leakage model, using the model proposed by Germanopoulos, as implemented in the pressure-driven analysis [5]. Both hydraulic model and calibration procedure are included in the WDNetXL system.

Thereafter, the procedure is applied on the WDN model for Oppegård municipality (Norway), where the calibration of background leakage model parameters is of primary importance for two main reasons: (i) changing pipe hydraulic resistances within technically consistent values would not produce sensible variation of the WDN pressure regime since it is quite high due to system oversize and the wide range of variations of ground elevation; (ii) the main purpose of WDN modelling was to support decisions on planning pressure control strategies by assessing the leakage reduction achievable using different alternatives, thus the accurate estimate of hydraulic resistances would have a limited impact on such analyses.

The application on Oppegård WDN shows that the grouping of pipes according to similar propensity to leak as well as the introduction of distance from prior parameter values in the multi-objective optimization strategy are of key importance in calibrating the leakage model parameters, based on information which are actually available. In fact, the strategy permitted to define prior values of β_k for each cohort of pipes based on existing data/information provided by the water utility at both global level (i.e. water balance) and in terms of expected leakages at each node (e.g. based on some assumptions of the water utility). On the other hand, minimizing the distance from such priors while minimizing the mismatching from observed data, also permitted to achieve technically consistent results in spite of the possible inconsistency between observed real data and the boundary conditions (e.g. customers' demand values and patterns) used for the EPS.

These features of the proposed methodology permitted to develop a *phenomenological* model which is currently adopted for supporting the plan of pressure control strategies for leakage reduction, whose main purpose is to compare the background leakage reduction achievable assuming different alternative technical solutions.

Forthcoming research in this field is expected to include other models for leakages, even different from the Germanopoulos' one, maybe including multiple parameters to be estimated. Nonetheless, this work emphasizes that implementing more complex models, suited to tackle new technical needs, requires also improved WDN monitoring and updated system information.

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