Relevance of hydraulic modelling in planning and operating real-time pressure control: case of Oppegård municipality

Luigi Berardi, Antonietta Simone, Daniele B. Laucelli, Rita M. Ugarelli and Orazio Giustolisi

ABSTRACT

Technical best practices recommend pressure control as an effective countermeasure to reduce leakages in water distribution networks (WDNs). Information and communication technologies allow driving pressure reducing valves (PRVs) in real-time based on pressure observed at remote control nodes (*remote real-time control – RRTC*), going beyond the limitations of *classic* PRV control (i.e. with target pressure node just downstream of the device). Nowadays, advanced hydraulic models are able to simulate both *RRTC*-PRVs and *classic* PRVs accounting for unreported and background leakages as diffused pressure-dependent outflows along pipes. This paper studies how such models are relevant to support pressure control strategies at both planning and operation stages on the real WDN of Oppegård (Norway). The advanced hydraulic model permits demonstration that *RRTC*-PRVs in place of existing c*lassic* PRVs might reduce unreported and background leakages by up to 40%. The same analysis unveils that advanced models provide reliable evaluation of leakage reduction efforts, overcoming the inconsistencies of lumped indexes like the Infrastructure Leakage Index (ILI). Thereafter, the model allows comparison of three strategies for the real-time electric regulation of PRVs in some of the planned scenarios, thus supporting real-time operation of *RRTC*-PRVs.

Key words | leakages, pressure control valves, remote real-time control, water distribution networks

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INTRODUCTION

The World Bank estimated that non-revenue water (NRW) costs utilities about 14×10^9 US$ per year (Kingdom *et al.* 2006), while the World Economic Forum reported water crisis as a top impact global risk (World Economic Forum 2015). Reducing real losses from water distribution networks (WDNs) is a major management issue that has many operational benefits including the improvement of system hydraulic capacity, the increase of asset longevity, the saving of water resources and, ultimately, the reduction of the carbon footprint for water abstraction, treatment and pumping (European Commission 2013).

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Technical literature classifies real water losses as *bursts* and *background leakages* (Lambert 1994). Such definition is consistent with the aim of allocating budget for active leakage control based on the International Water Association (IWA) global water balance.

Pipe *bursts* represent large water outflows that might cause severe disruptions and third-party damage. Although *burst* leaks can be considered as accidental events, much of the literature investigated external factors driving their occurrence (e.g. Lei & Saegrov 1998; Kleiner & Rajani 2001). Major *burst* leaks that have relevant impact on pressure and water supply service are usually *reported* to water utilities and repaired in a short time. Other bursts are *unreported* to water utilities and run until detected through active leakage control actions. The main approach to minimize the impact of pipe *bursts* is to improve system mechanical reliability, e.g. increasing the effectiveness of isolation valve systems (Walski 1993; Yazdani & Jeffrey 2012), and implementing strategies for prompt *detection*, *localization* and *repair* of new leaks (e.g. Berardi *et al.* 2014; Romano *et al.* 2014).

Background leakages represent small outflows from joints, fittings or holes/cracks along the pipeline and often happen along connections to private properties. The IWA (Farley & Trow 2003) reports *background leakages* as leaks 'with flow rates too low to be detected by an active leakage control campaign'.

From an asset condition perspective, *background leakages* accelerate pipe deterioration as a combination of various concurrent phenomena (e.g. Kleiner & Rajani 2002) including a wide spectrum of local leaking conditions leading to major bursts.

From a hydraulic perspective, both *unreported* and *background* leakages are pressure-dependent components of real water losses that do not cause abrupt changes in WDN hydraulic behaviour. For these reasons, they run for a long time characterizing normal working conditions and have major volumetric effects on global WDN mass balance (e.g. on annual operating cycle). Therefore, *unreported* and *background* leakages cannot be neglected in hydraulic modelling aimed at supporting WDN management decisions and we will designate the summation of these two components of leakages as *volumetric real losses*. The increase of flow rate due to *volumetric real losses* is a relevant indicator for 'asset management' because high *volumetric real losses* relate to asset deterioration and/or pressure higher than the values required for a correct and reliable service.

The main approaches that are advised to reduce *volumetric real losses* are: asset rehabilitation, leakage detection programmes, pressure control and districtualization (Laucelli *et al.* 2017).

Asset rehabilitation is known to provide *medium–long* term solutions (e.g. Alvisi & Franchini 2009; Giustolisi & Berardi 2009), and requires higher investments and careful selection of pipes to be replaced.

Pressure control is recommended (e.g. Vairavamoorthy & Lumbers 1998; Farley & Trow 2003) as a *short–medium* term best practice to reduce *volumetric real losses*. It aims at reducing pressure while preserving adequate water supply service conditions. In addition, pressure control strategies allow reduction of the rate of rising of *reported leaks* also in the long term (Girard & Stewart 2007).

This work looks at pressure control via pressure reducing valves (PRVs), accounting for both *classic* (local) and *remote real-time control* (*RRTC*) strategies.

Classic (local) control of PRVs consists of modulating the valve opening to maintain a target pressure at a control node just downstream of the devices. Nonetheless, the change of customers' water demands over time causes a change of head losses through the WDN. This, in turn, requires a time-pattern of target pressure values of the *classic* PRV in order to avoid insufficient pressure conditions or excessive pressure at peak demand and low demand hours, respectively. A number of authors proposed alternative strategies to optimize the time modulation of *classic* PRVs. The early work by Sterling & Bargiela (1984) proposed an optimal valve control to minimize the network overpressure, although it did not explicitly include the leakage term as water outflow. Vairavamoorthy & Lumbers (1998) accounted for leakages and optimized time modulation of PRVs, relaxing the constraint of maintaining a target pressure in a few nodes, thus further reducing leakages. Nonetheless, they did not account for any pressure-demand relationship in their problem formulation. The work by Ulanicki *et al.* (2000) first investigated the possibility of using optimized *predictive* or *feedback* control strategies. The *predictive* control uses a hydraulic model to optimize the valve scheduling based on predicted/measured total demand. The *feedback* control is distinguished as *centralized* (the settings of each valve is computed as a function of flow measurements at all PRVs) or *decentralized* (the settings of each valve is computed as a function of flow measurements at the same PRV); in both cases the controller is based on off-line simulation of the WDN. Other works mainly proposed strategies for the optimal location of PRVs and relevant setting modulation (e.g. Araujo *et al.* 2006; Ulanicki *et al.* 2008; Abdel & Ulanicki 2010). All such works used a traditional hydraulic simulator, e.g. based on EPANET2 solver (Rossman 2000).

RRTC of PRVs consists of modulating the valve opening according to a target pressure at a control node, which can be in a remote position from the PRV. Indeed, information and communication technologies (ICT) in the water sector enables transmission of pressure observations sampled from any node in the WDN to a programmable logic control (PLC) unit that electrically modulates the opening of the valve in order to maintain the desired target pressure at the control node. *RRTC*-PRV strategies are more reliable and effective in controlling pressure and reducing leakages than *classic* PRVs (Giustolisi *et al.* 2017). Nonetheless, *RRTC*-PRVs require careful planning of both valve location and control node in order to ensure controllability. In addition, *RRTC*-PRVs require effective and efficient strategies for the electric regulation of the PRVs that have to be implemented at PLC units to drive valve opening over time.

Most of the technical literature on *RRTC*-PRVs reports strategies for the real-time modulation of the valve opening degree in terms of the transfer function to be implemented at PLC units (e.g. Campisano *et al.* 2010; Creaco & Franchini 2013). Recently, Giustolisi *et al.* (2017) compared three alternative strategies for the electric regulation of *RRTC*-PRVs, using the shutter opening degree (SD), the valve hydraulic resistance (RES) and the valve head loss (HL) as control variables, respectively. Other works analyse optimal setting and/or location of *classic* PRVs (e.g. Prescott & Ulanicki 2008; Creaco & Pezzinga 2014).

Unfortunately, no literature contributions provide a comprehensive view on using WDN hydraulic models to support planning and operation of *RRTC*-PRVs for leakage reduction purposes. The main causes of this gap stem from the limitations of traditional hydraulic models, which do not provide reliable analyses to support such decisions in real contexts.

The present work demonstrates that advanced WDN hydraulic models, incorporating pressure-dependent modelling of *volumetric real losses* (e.g. Giustolisi *et al.* 2008) and the simulation of *RRTC*-PRVs are highly relevant to support planning pressure control schemes and selection of efficient strategies for electric regulation of the PRVs.

The next two sections discuss the representation of *reported bursts, unreported* and *background* leakages as well as the differences between *classic* (local) and *RRTC* control of PRVs in advanced WDN hydraulic models. Thereafter, the different assumptions behind WDN hydraulic modelling for *planning* and *operational* purposes are discussed and demonstrated using the real WDN of Oppegård (Norway).

For *planning* purposes, the analyses allow comparison of the current pressure control scheme, based on *classic* PRVs, with a few alternative scenarios including *RRTC*-PRVs. Results demonstrate that *RRTC*-PRVs in place of some *classic* PRVs enables reduction of the leakage volume by up to 40%. Moreover, the results of the advanced WDN hydraulic model reveal that the IWA Infrastructure Leakage Index (ILI) (e.g. Farley & Trow 2003) can be misleading for tracking progresses in leakage management.

For operational purposes, the advanced hydraulic model allows comparison of three alternative strategies for the electric regulation of *RRTC*-PRVs in one of the new planned scenarios.

BURSTS AND BACKGROUND LEAKAGES IN ADVANCED WDN HYDRAULIC MODELS

Reported bursts, can be represented in WDN hydraulic models as *free orifices* at failure points (e.g. major holes, joint displacements, cross-sectional or longitudinal cracks). Since their location is known when they occur, *reported bursts* can be included as additional nodes in the original WDN hydraulic model in order to assess the impact of burst failure. The outflow from *burst* nodes follows a pressure-discharge relationship inspired by the Torricelli law, where the orifice discharge area depends on pipe material, orifice shape and local (node) pressure. Equation (1) reports the leakage outflow d_i^{leak} from a single *burst* node (Giustolisi & Walski 2012):

$$d_i^{leaks}(P_i) = \begin{cases} a_i^{leaks}(P_i)\sqrt{P_i} = \beta_i P_i^{\alpha_i} & P_i > 0 \\ 0 & P_i \le 0 \end{cases} \quad (1)$$

where i = subscript of the ith node; P_i = model pressure at the ith node; d_i^{leak} = leakage outflow at the ith node; $a_i^{leak}(P_i)$ = outflow coefficient depending on pressure; β_i = coefficient of the burst-leakage model; α_i = exponent of the burst-leakage model whose value is larger than 0.5 to account for direct pressure-area variation.

From a technical perspective, accounting for *reported bursts* in WDN hydraulic simulation allows analysis of abnormal scenarios where the *burst* location is known or assumed (e.g. analysis of past events; study of system reliability under unexpected events; *leakage* pre-localization procedures). *Vice versa, reported bursts* are not relevant when WDN hydraulic analysis aims at supporting WDN management decisions pertaining to normal conditions (i.e. most of the service time of the system).

As mentioned above, *unreported* and *background leakages* (*volumetric real losses*) characterize the normal WDN working conditions and have major volumetric effects on the global WDN mass balance. Since the exact locations of *volumetric real losses* are not known, they can be represented as diffused outflows dependent on the average pipe pressure as in Equation (2), where P_i and P_j are the mean pressures at ending nodes i and j of the kth pipe.

$$P_{k,mean} = \frac{P_i + P_j}{2} \tag{2}$$

In recent years, the most widely adopted models for such *volumetric real losses* resort to Germanopoulos (Germanopoulos 1985; Germanopoulos & Jowitt 1989) or to the fixed and variable area discharge (FAVAD) approach (May 1994).

Germanopoulos' formulation assumes that outflow of *unreported* and *background* leakages (d_k^{leaks}) from the kth pipe in the network depends on $P_{k,mean}$ as in Equation (3),

$$d_k^{leaks}(P_{k,mean}) = \begin{cases} a_k^{leaks}(P_{k,mean})\sqrt{P_{k,mean}} = \beta_{1,k} L_k P_{k,mean}^{\alpha_k} & P_{k,mean} > 0 \\ 0 & P_{k,mean} \leq 0 \end{cases} \tag{3}$$

where L_k is the length of the kth pipe and the exponent α_k is larger than 0.5 to account for pressure-area variation.

Van Zyl & Cassa (2014) recently investigated the FAVAD concept, proposing the formulation in Equation (4).

$$d_k^{leaks}(P_{k,mean}) = \begin{cases} \beta_{1,k} L_k P_{k,mean}^{0.5} + \beta_{2,k} L_k P_{k,mean}^{1.5} & P_{k,mean} > 0 \\ 0 & P_{k,mean} \leq 0 \end{cases} \tag{4}$$

As parameters of the model of *volumetric real losses*, $b_{1,k}$ and α_k for Equation (3), or $\beta_{1,k}$ and $\beta_{2,k}$ for Equation (4), have to be calibrated for each kth pipe in the hydraulic model, in addition to the pipe hydraulic resistances and the pattern of customers' demands (e.g. Berardi *et al.* 2017). Although model calibration is outside the scope of this work, the linear relationship with respect to $\beta_{1,k}$ and $\beta_{2,k}$, makes the model in Equation (4) much better suited for calibration than the power model in Equation (1).

From a WDN management perspective, coefficients $[\beta]_{1,k}$ or $\{[\beta]_{1,k}, [\beta]_{2,k}\}$ represent *volumetric real losses* outflow per unit pipe length and unit average pressure. Therefore, they encompass the deterioration effects of many factors like age, material, diameter, external loads, pressure regime, etc. Laucelli & Meniconi (2015) compared the formulations in Equations (3) and (4) reporting also the relationships among the parameters, observing that in Equation (3) the parameter α_k entails the pressure-area variation, while the parameter $\beta_{1,k}$ accounts for those outflows with low head-area slopes, as in the first addendum in Equation (4).

It was demonstrated (Giustolisi *et al.* 2016) that the representation of *volumetric losses* as concentrated outflows (i.e. hydrants) at pipe end nodes (i.e. depending on nodal pressure values) is not consistent with the actual hydraulic behaviour of the pipe because it returns dissimilar demand and pressure distribution in the network. In addition, such a modelling assumption is not effective for supporting the understanding of the system components (mains or connection to properties) that actually generate losses, e.g. for pipe rehabilitation purposes. In fact, concentrating leakages at nodes prevents simulation of the effects of replacing one single pipe (i.e. making null its leakage parameters) on WDN behaviour as a whole.

PLANNING AND OPERATING PRVS: *CLASSIC* VS. *RRTC*

Effective pressure management in a WDN should guarantee sufficient pressure for correct supply service to customers while minimizing leakages that have major volumetric effects on the WDN water balance. PRVs aim at minimizing the excess of pressure over that required for correct supply service by automatically *modulating* the valve opening. From a hydraulic modelling standpoint, a PRV introduces a minor head loss ΔH_{PRV} varying over time t as in Equation (5):

$$\Delta H_{PRV}(t) = k_{ml}(t)Q^2_{PRV}(t) \tag{5}$$

k_{ml} is the minor head loss coefficient, which depends on the opening degree of the valve. Q_{PRV} is the flow through the valve, which depends on the required demands at nodes and *volumetric real losses* along the flow paths downstream of the valve. Each water demand component varies over time t according to its specific type (e.g. human requests, volume controlled orifices, and so on, as in Giustolisi & Walski (2012)). *Volumetric losses* are driven by pressure through the network (as reported in the previous section), which depends on head losses, i.e. on time varying demands.

In the case of control of a *classic* PRV, k_{ml} changes in order to maintain a target pressure $P_{target}(t)$ just downstream of the valve,

$$P_{target}(t) = P_{up}(t) - k_{ml}(t)(Q^D_{PRV}(t) + Q^L_{PRV}(t))^2 \tag{6}$$

$P_{up}(t)$ is pressure just upstream of the valve (i.e. $\Delta H_{PRV}(t) = P_{up}(t) - P_{target}(t)$); $Q^L_{PRV}(t)$ is the portion of $Q_{PRV}(t)$ related to *volumetric losses*; $Q^D_{PRV}(t)$ is the portion related to all water demand components different from leakages.

In order to guarantee adequate water supply service conditions, $P_{target}(t)$ should increase as water supply downstream of the valve (i.e. $Q_{PRV}(t)$) increases. In fact, larger flows generate larger head losses through the network, thus requiring higher upstream pressure to avoid pressure deficient conditions. Therefore, $P_{target}(t)$ just downstream of the valve should be varying over time depending on the hydraulic behaviour of the network.

In more detail, $P_{target}(t)$ should depend on the required water demand $Q^D_{PRV}(t)$ of nodes in the areas which are reached by flow paths passing through the PRV and the outflow from diffuse *leakages* $Q^L_{PRV}(t)$ that, in turn, depend on pressure (i.e. $P_{target}(t)$). On the one hand, setting a value of $P_{target}(t)$ lower than the required hydraulic capacity, (i.e. depending on $Q_{PRV}(t) = Q^D_{PRV}(t) + Q^L_{PRV}(t)$), means to incur into insufficient pressure. On the other hand, setting a conservative high value of $P_{target}(t)$ would result into inefficient reduction of *volumetric losses*.

Overall, planning a reliable P_{target} over time is difficult because it depends on current network behaviour. In field application, a single reliable value of P_{target} is generally set for night conditions only, leaving the valve open during the day. Sometimes, a value of P_{target} is also set for daytime conditions. Nonetheless, the need for avoiding pressure deficient conditions often motivates the selection of conservative P_{target} values that make the pressure control inefficient for leakage reduction.

In *RRTC*-PRVs, $P_{target}(t)$ can be set at any internal node of the hydraulic system, even far from the PRV location. Therefore, Equation (6) becomes,

$$P_{down}(t) = P_{up}(t) - k_{ml}(t)(Q^D_{PRV}(t) + Q^L_{PRV}(t))^2 \tag{7}$$

where $P_{down}(t)$ is the pressure just downstream of the valve ($\Delta H_{PRV}(t) = P_{up}(t) – P_{down}(t)$).

The controlling node of the PRV is the *critical* node from a hydraulic perspective, i.e. the node with the lowest difference between the required pressure for correct service and the pressure expected over time without pressure control. Since the required pressure for correct service depends on nodal elevation, building heights and required minimal residual pressure (e.g. by regulations), it does not vary over time (Giustolisi & Walski 2012). The identification of the *critical* node is not difficult if the spatial variation of demands is not significant. Sometimes, there are few nodes close to *critical*, thus a technical expedient is to set the constant target pressure in one *critical* node equal to the pressure for correct service (i.e. $P_{target}(t) = P_{crit\text{-}serv}$) plus, possibly, a small value to be conservative over other nodes close to *critical*.

Let us assume $P_{crit}(t)$ as the pressure observed at a *critical* node at time t, and $\Delta H_{crit}(t) = P_{crit}(t) - P_{crit\text{-}serv} = P_{crit}(t) - P_{target}$. Considering Equation (5), the valve has to be modulated such that $\Delta H_{crit}(t) = \Delta H_{PRV}(t)$ over time in order to achieve $P_{crit}(t) = P_{crit\text{-}serv}$. Therefore, *RRTC*-PRVs ask for real-time transfer of $\Delta H_{crit}(t)$ to a PLC using communication technologies (e.g. using radio, Global System Mobile – GSM protocols, etc.). Therefore, the PLC has to drive an actuator to modulate the valve opening in order to achieve $\Delta H_{PRV}(t) = \Delta H_{crit}(t)$.

ADVANCEMENTS IN WDN HYDRAULIC MODELS FOR SUPPORTING LEAKAGE REDUCTION

Planning pressure control strategies in WDNs is a complex technical problem, which requires investments (i.e. for purchasing and installing devices) and modifications of the existing WDN in terms of topology and hydraulic functioning (e.g. by closing valves to increase controllability). Accordingly, water utilities need careful evaluation of alternative pressure control scenarios and prediction of reduction of *unreported* and *background leakage* volume in order to plan investments and schedule works.

Unfortunately, traditional WDN models (e.g. based on EPANET2 – Rossman (2000)) are not able to support such kinds of analyses because of some main limitations. (a) They do not allow the *volumetric leakage* model as pressure-dependent outflow distributed along pipes. (b) They do not allow WDN analysis under pressure deficient conditions (which might happen while testing alternative pressure control scenarios). Indeed, the hydraulic model is cast as *demand-driven* and insufficient pressure conditions do not affect water delivered to customers. (c) They only allow *classic* PRVs, preventing evaluation of the potentialities of *RRTC*-PRVs before their implementation. The latter limitation has motivated heuristic approaches for planning and operating *RRTC*-PRVs so far. (d) Traditional hydraulic models rely on the assumption of immutable topology. Such a hypothesis results in numerical expedients to emulate the valve closure, without explicitly accounting for the possible disconnection of WDN sub-portions.

Advanced WDN models allow the representation of all elements as in traditional ones, but overcome all the above-mentioned major limitations, thus being of direct relevance to support pressure control. In fact, they entail *pressure-driven* analysis, accounting also for the *volumetric leakage* model (i.e. as in Equation (3) or (4)), allowing analysis of WDN functioning under pressure deficit scenarios. In addition, pressure control devices (i.e. PRVs) can be controlled from any node in the network and, in the case of PRV shutdown, the actual current topology is automatically identified, i.e. by removing the disconnected WDN portion from the model. The advanced model used in this work (Giustolisi *et al.* 2008, 2011, 2017) includes all such features.

HYDRAULIC MODELLING ASSUMPTIONS TO SUPPORT PLANNING AND OPERATION OF *RRTC*-PRVS

The analysis of *RRTC*-PRVs for *planning* purposes aims at studying alternative *RRTC*-PRV configurations accounting for: the location of valves and *critical* nodes; the controllability over time; the main functioning of the valves in order to detect possible interference among devices; and the reduction of *volumetric losses* subsequent to pressure control. Accordingly, the hydraulic modelling to support *planning* pressure control schemes does not account for the temporary effects of valve adjustments (lasting for a few minutes). Rather, the main assumption here is that the target pressure P_{target} at critical nodes is reached *instantaneously* and is kept constant over each modelling time step (e.g. 1 hour). In fact, for *planning* purposes, the model aims at representing the average functioning of the WDN during each simulation time step, which is sufficient to compare alternative pressure control schemes. The valve head loss (i.e. $\Delta H_{PRV}(t)$) to achieve P_{target} at a critical node is assumed the same over the whole simulation time step. Such an assumption is consistent with the hypotheses for steady-state simulation in the WDN assumed modelling paradigm (e.g. Todini & Pilati 1988; Giustolisi *et al.* 2008) and enables performing of the extended period simulation (EPS) over a typical operating cycle (e.g. 24 hours).

The analysis of *RRTC*-PRVs for supporting system *operations* follows the idea that theoretical and numerical studies should forerun the implementation of new operational strategies in real systems. Such an approach aims at increasing the awareness of the technical alternatives before taking operational decisions. In this framework, advanced WDN hydraulic models enable comparison among different real-time control strategies that can be implemented at PLC units for the electrical regulation of *RRTC*-PRVs (Giustolisi *et al.* 2017). Indeed, the effectiveness and efficiency of each control strategy are influenced by many factors such as: (i) the valve curve; (ii) the selected control variables to drive valve opening that might require additional flow/pressure measurements; and (iii) the transfer function to set the valve opening based on pressure reading at a *critical* node over time.

Moreover, the adjustment of a PRV cannot instantaneously achieve the critical pressure level $P_{crit}(t) = P_{crit\text{-}serv}$ at the *critical* node. In fact, during the adjustment time step (i.e. valve modulation towards the set opening degree) the hydraulic system behaviour changes, therefore $P_{crit}(t)$ and ΔH_{crit} (t) change. In addition, abrupt valve manoeuvres must be avoided in order to prevent unsteady flow instabilities (e.g. Brunone & Morelli 1999; Prescott & Ulanicki 2008; Meniconi *et al.* 2015). All such circumstances require accounting for the control time step Tc (i.e. of few minutes) and for the maximum valve displacement $\Delta\alpha$ (i.e. as the product between Tc and the maximum shutter velocity to avoid unsteady conditions $v_{max\text{-}\alpha}$) during each simulation run.

For these reasons, the hydraulic analysis of *RRTC*-PRVs for *operational* purposes refers to consecutive snapshots of WDN behaviour, each equal to the control time interval Tc. The hydraulic model is used to analyse and compare the efficiency of various pressure control strategies over a typical WDN operating cycle (e.g. 24 hours).

Although the analysis for *operative* purposes refers to time steps (i.e. Tc) shorter than for *planning* purposes (i.e. 1 hour), the hypotheses for steady-state simulation are still valid since PRV manoeuvre spans over a few minutes, due to the constraints on the maximum valve displacement $\Delta\alpha$. In order to preserve the hydraulic consistency of the analysis, the customer-required demand, which normally refers to longer time steps (e.g. $\Delta T = 1$hour) can be linearized over each Tc simulation interval (Giustolisi *et al.* 2017); the model of *volumetric losses* reported in Equations (3) and (4) still holds.

REAL CASE STUDY: SUPPORTING PRESSURE CONTROL IN OPPEGÅRD WDN

This section demonstrates the effectiveness of using advanced WDN hydraulic models to support *planning* of pressure management strategies and *real-time operation* of *RRTC*-PRVs in a real scenario.

Planning pressure control in real contexts leverages *prior* analyses of the systems, including the empirical knowledge of high-pressure zones as well as practical constraints. For example, the knowledge of existing vaults/manholes might reduce the investment needed for installing new valves. In addition, the *planning* of PRVs should analyse multiple scenarios where devices will be progressively installed accounting for PRVs and gate valves that already exist. In fact, water utilities are usually reluctant to switch from existing operations to completely different new control scenarios. Such abrupt changes require larger investments and, more importantly, their implementation has uncertain effects on WDN functioning, with possible failure of water supply service to customers. Following a conservative and pragmatic approach, new pressure control schemes should be analysed and gradually implemented starting from the current WDN configuration. Moreover, this approach allows updating the hydraulic models as soon as new field data are available, allowing simulation of the impact of alternative scenarios in a fully controlled environment. This, in turn, increases the credibility and acceptability of innovations.

The analysis herein refers to a portion of the WDN serving the North-West area of Oppegård municipality (Norway). The analysis pertains to alternative strategies to reduce real water losses by improving pressure control.

Oppegård WDN (Figure 1) extends for about 129 km, with elevation ranging from 40 m to 180 m a.s.l. Current WDN operation combines pumping, in order to provide sufficient pressure in high elevation areas, and pressure reduction, using *classic* PRVs, in order to limit pressure excess in lower zones. Pipe diameters are quite large in order to fulfil firefighting requests (minimum pressure of 30 m has to be guaranteed everywhere in the network). Such a condition, in combination with the peculiar elevation, results in very high pressure in some areas, even exceeding 120 m. Irrespective of the customers' water demand pattern, the pressure regime is almost constant over the day. The North-West Oppegård (rectangle in Figure 1) is an area where the municipality is looking for more effective pressure control strategies.

Figure 2 shows the elevation of North-West Oppegård as well as the location of the nine PRVs (white triangles) that are currently installed to control pressure. Such valves entail *classic* local pressure control, where target pressure (P_{target}) ranges between 35 m and 70 m immediately downstream of the valves. Currently such P_{target} values are kept constant over the operating cycle for each valve because of the negligible effect of customers' demand on the pressure regime.

The advanced hydraulic model for Oppegård WDN was preliminarily built and calibrated (Berardi *et al.* 2015). The

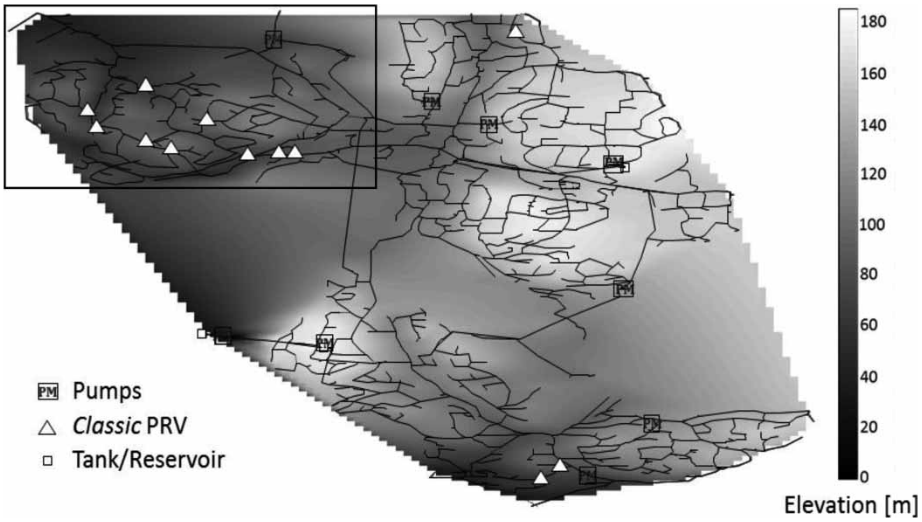

Figure 1 | Oppegård WDN layout and elevation.

exponent α_k in Equation (3) was assumed equal to 1.0 for all pipes and the average value of $\beta_{1,k}$ was 4.9×10^{-9}. Figure 3 shows the pressure surface in North-West Oppegård, drawn by interpolating the nodal pressure values obtained from the WDN model under the existing pressure control scenario.

Figure 4 reports the water delivered to customers in North-West Oppegård over 24 hours (i.e. users' demand pattern as provided by the water utility) and the *volumetric leakages* as from Equation (3) in the original configuration (black bars). Other bars refer to *volumetric leakages* in four alternative scenarios to be discussed in the next section.

As expected from the roughly constant pressure regime, the *volumetric leakages* (referring to the scenario '*ORIGINAL*') remain almost constant during the day with values of about 40 m^3/h.

Planning pressure control scenarios

The enhanced WDN hydraulic model is used to analyse alternative pressure control scenarios involving both

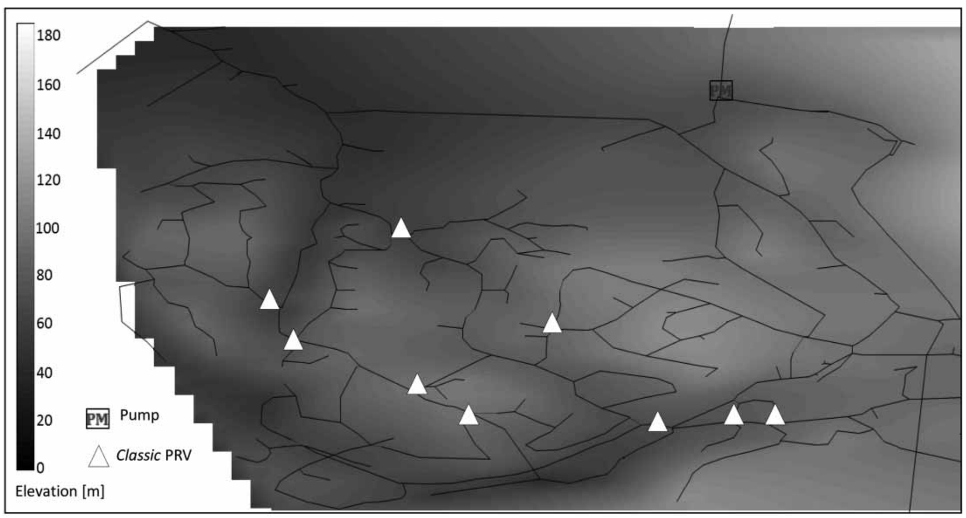

Figure 2 | North-West Oppegård WDN layout and elevation.

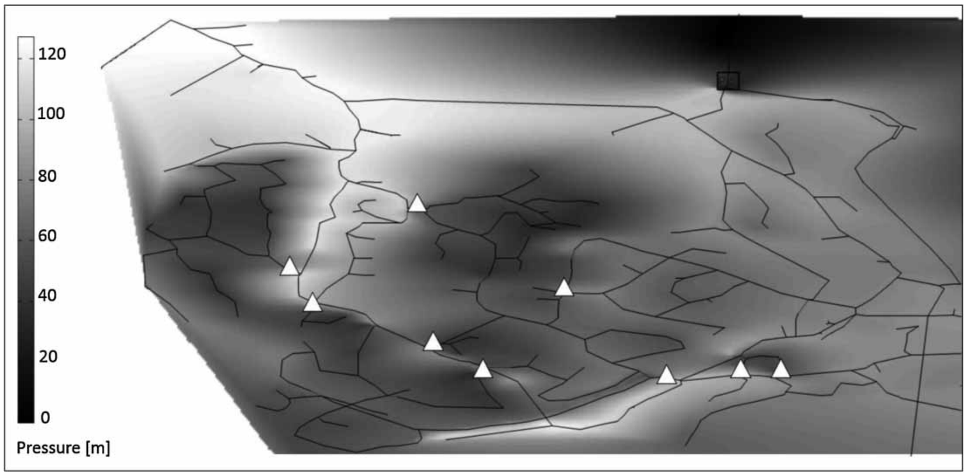

Figure 3 | Pressure surface in North-West Oppegård (WDN model).

existing *classic* PRVs and new *RRTC*-PRVs. The analysis of various scenarios involving a gradually increasing number of *RRTC*-PRVs intends to demonstrate how WDN models enable the increase of technicians' awareness about such new control schemes. This, in turn, aims at overcoming the empirical approaches used so far. For the sake of the brevity, this work reports four alternative scenarios entailing different numbers and locations of valves.

Scenario 1 (Figure 5 (1)) consists of seven PRVs, thus two less than the nine currently installed. Three are new *RRTC*-PRVs (black triangles) and four are *classic* PRVs already installed (black squares). Both location

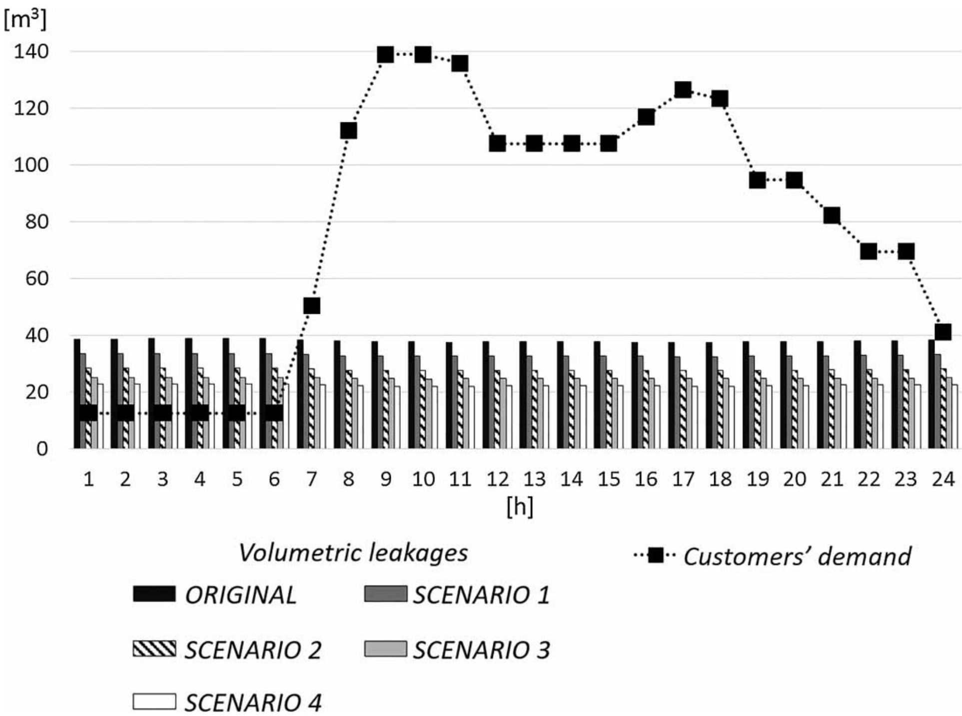

Figure 4 | Water demand components in North-West Oppegård: customers' demand and *volumetric leakages* under various pressure control scenarios (Scenario 1: 3 *RRTC*-PRVs; Scenario 2: 3 *RRTC*-PRVs; Scenario 3: 4 *RRTC*-PRVs; Scenario 4: 5 *RRTC*-PRVs).

and P_{target} values of the existing *classic* PRVs are unchanged.

White circles represent the *critical* nodes controlled by the relevant *RRTC*-PRVs. In order to guarantee that the PRVs actually control the intended *critical* nodes, the pressure control areas are delimited by closing some existing gate valves and are shadowed in Figure 5. From a WDN management perspective, this solution allows reduction of current leakages by 13% of the water volume currently lost from North-West Oppegård (see Figure 4).

Scenario 2 (Figure 5 (2)) is quite similar to *Scenario 1*, while the valve located in the eastern part of the analysed area actually allows control of a large portion of the network, shadowed in grey, which is at very low elevation. This configuration results in pressure reductions up to 40 m and allows reduction of the lost water volume of about 27% of the current *volumetric leakages* in North-West Oppegård (see Figure 4).

Scenario 3 (Figure 5 (3)) consists of eight PRVs (one less than the number currently installed). It has one more *RRTC*-PRVs than those installed in *Scenario 2* (i.e. four *RRTC*-PRVs in total). The new *RRTC*-PRV (black circle in Figure 5 (3)) allows further pressure reduction in the relevant controlled area, resulting in about a 35% reduction of real losses in the current scenario (see Figure 4).

Scenario 4 (Figure 5 (4)) consists of adding another *RRTC*-PRV and changing the configuration of the gate valves. This new scenario further improves pressure control in the northern part of the analysed area, which is at low elevation. Pressure reduction in the extreme nodes is about 60 m. This means that the same number of valves (i.e. nine) as in the current configuration, but including five *RRTC*-PRVs, allows reduction of leakages in North-West Oppegård by more than 40% (see Figure 4).

Due to the pressure exceeding the minimum required for correct service, the customers' demand in Figure 4 does not change among the analysed PRVs configurations. Vice versa, *volumetric leakages*, which are almost invariant over the 24 hours, change according to the pressure control scheme adopted, which involves different numbers of *RRTC*-PRVs. The same figure also shows that, due to the pressure regime, during the night the *volumetric losses* are still higher than the (expected) water consumption. Further reduction would be possible at the cost of additional *RRTC*-PRVs, controlling WDN sub-areas. Such additional configurations are neglected herein for the sake of brevity.

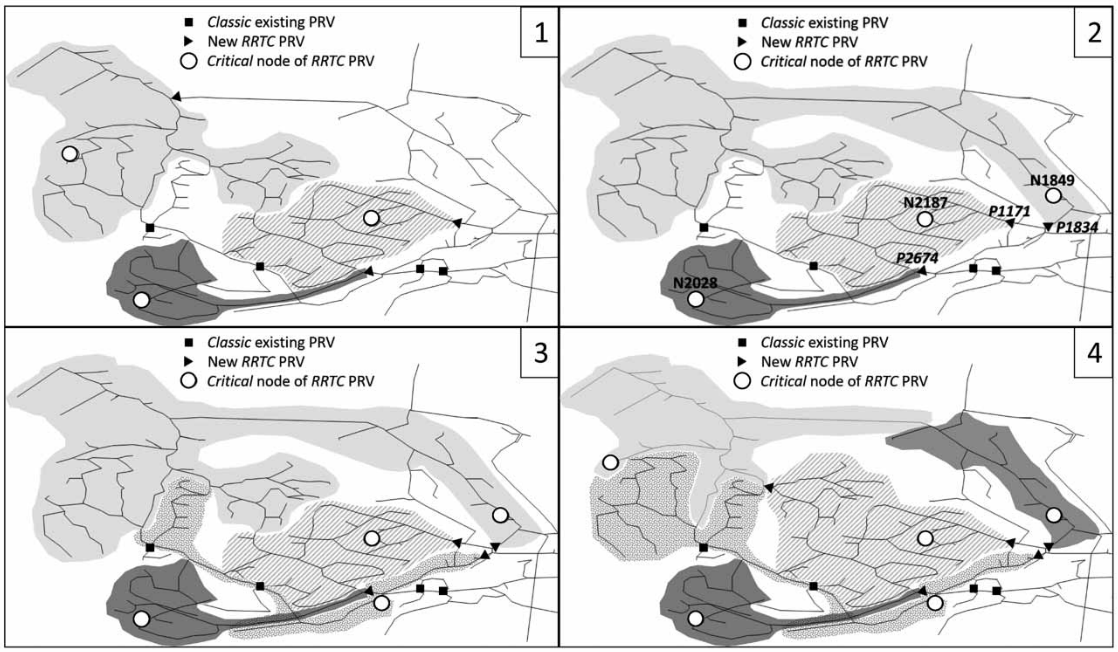

Figure 5 | Planning scenarios of pressure control.

Figure 6 reports the water lost from *volumetric losses* and the predicted water savings on an annual basis, for each alternative scenario (i.e. numbers of *RRTC*-PRVs). This figure provides a synthetic tool to support water utilities in taking decisions about the effectiveness of implementing new pressure control strategies, starting from current '*ORIGINAL*' scenario.

Figure 6 also reports the value of the IWA ILI (e.g. Farley & Trow 2003) in North-West Oppegård. ILI is computed as the ratio between the Unavoidable Annual Real Losses (UARL), which is proportional to the average system pressure (P_{avg}), and the Current Annual Real Losses (CARL). For each scenario, the hydraulic model computes the average pressure (P_{avg}) for the analysed area and the CARL coincides with annual volume of *volumetric losses*. The UARL $[m^3/year] = (6.57 \cdot Lm + 0.256 \cdot Nc + 25 \cdot Lp) \cdot P_{avg}$, where the term in brackets does not change among the analysed configurations since it depends on the length of mains (Lm [km]), the number of service connections (Nc) and the total length of private pipeline to customer meters (Lp [km]).

Figure 6 shows that in moving from the current '*ORIGINAL*' condition to *Scenario 1* (i.e. 13% leakage reduction) the ILI does not change (i.e. ILI = 6.93), while in moving from *Scenario 2* to *Scenario 3* (i.e. 10% leakage reduction) the ILI slightly increases from 6.27 to 6.32. Similar results hold for other values of exponent α_k, both smaller and larger than 1.0, although not reported herein for the sake of brevity.

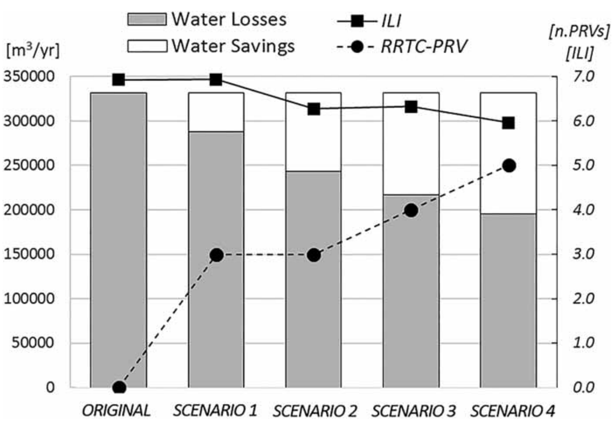

Figure 6 | Annual volumes of water lost and saved, number of RRTC-PRV and ILI for each pressure control scenario.

Such observations demonstrate that using the ILI to assess the leakage reduction achievements is not consistent with the expected hydraulic WDN behaviour. Consequently, the use of ILI for regulation purposes in the WDN sector would be misleading without the support of appropriate hydraulic modelling.

Supporting real-time operation of *RRTC*-PRVs

The analysis of *RRTC*-PRVs for supporting system *operations* aims at comparing three different strategies to control PRV opening based on pressure readings at remote control nodes. Such strategies designated as *HL*, *RES* and *SD* are detailed in Giustolisi *et al.* (2017) and are briefly summarized herein for the sake of completeness.

HL strategy takes the head loss across the PRV as the control variable: given the pressure deviation (ΔH_{set}) from the target set-point value observed at the critical node between time *t-Tc* and *t*, the strategy predicts the PRV head loss (ΔH_{PRV}) that has to be achieved during the next control interval (i.e. between t and $t + Tc$). This means that valve shutter moves with the maximum allowed velocity to avoid unsteady conditions, until the target ΔH_{PRV} value is observed across the valve using a differential pressure measurement.

RES strategy assumes the valve opening degree α (i.e. $\alpha = 0$: closed; $\alpha = 1$: fully open) as the control variable based on the valve curve. The predicted valve hydraulic resistance (i.e. $K_{ml}(t, t + Tc)$) to be achieved between t and $t + Tc$ depends on the ratio between the pressure deviation at the *critical* node (ΔH_{set}) and the valve flow (Q_{PRV}), both observed between *t-Tc* and *t*. As such, *RES* strategy requires the flow measurement at the PRV.

SD strategy also assumes the valve opening degree (α) as the control variable based on the valve curve and the pressure deviation at the *critical* node (ΔH_{set}). Differently from the *RES* strategy, it does not require additional flow/pressure measurements, but needs the calibration of a proportional gain (k_c) of the control function, which is assumed to be constant over time, irrespective of valve flow.

The hydraulic analysis is aimed at supporting the decision to purchase and install additional pressure/flow gauges at PRVs in order to implement *HL* or *RES*, rather than *SD*.

The analysis for operational purposes in North-West Oppegård is performed by subdividing the original simulation intervals ΔT (i.e. 60 min) into time steps of $Tc =$ 5 min. The EPS generates sequences of 12 snapshots of WDN behaviour into each hour. The customer-required demand over each Tc is obtained by linearizing the original demand values between consecutive ΔT. For the sake for the example, the analyses reported herein refer to *Scenario 2* in Figure 5 (2) and assume strategies *HL*, *RES* and *SD*. *SD* strategy assumes the proportional gain value ($k_c = 0.001$), which was the best performing as reported in Giustolisi *et al.* (2017) for the same system.

Figure 7 shows the results of the EPS in terms of pressure at *critical* nodes (left) and opening degree

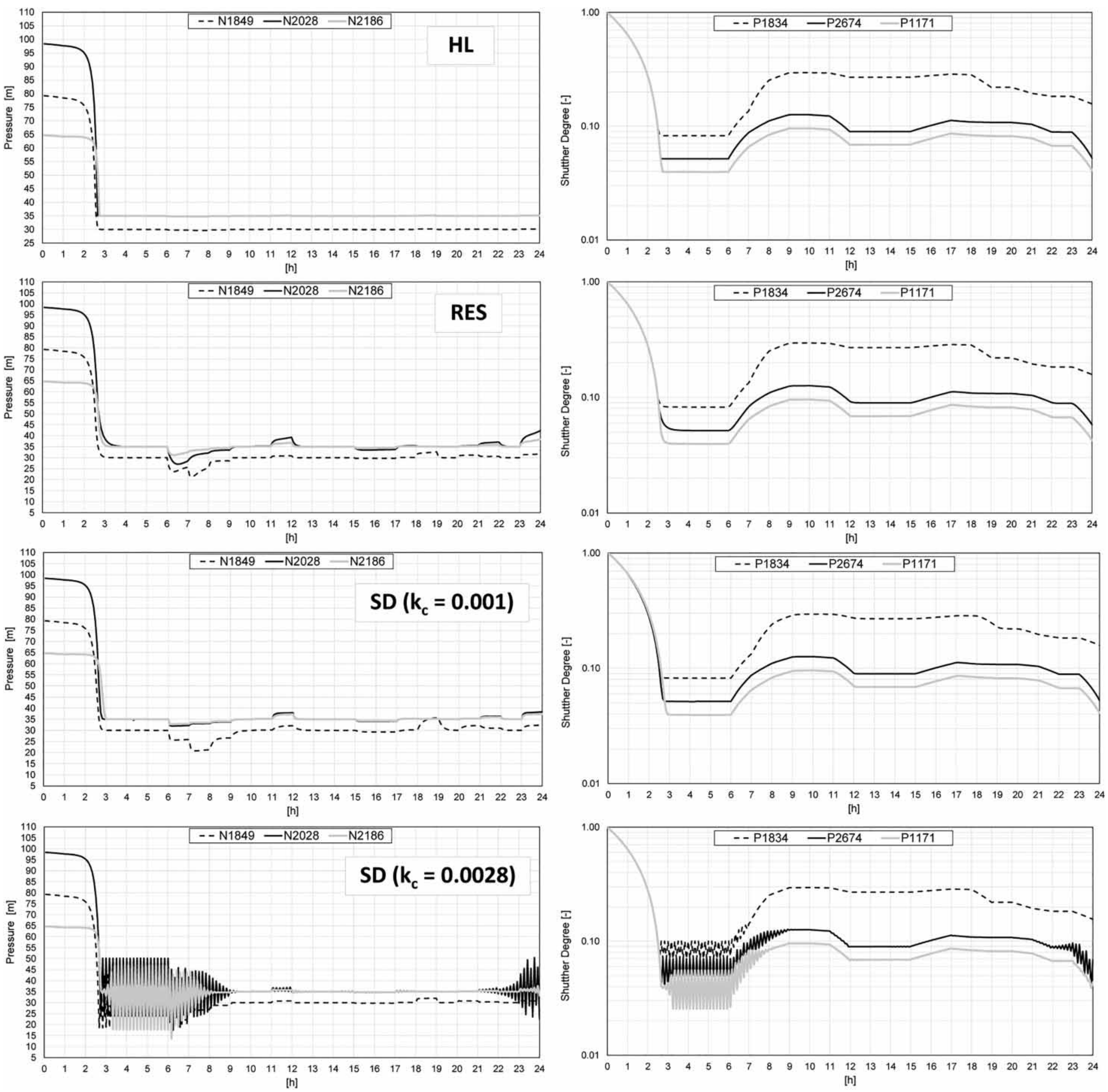

Figure 7 | EPS analysis of RRTC-PRVs *operation* using control strategies *HL*, *RES* and *SD* ($k_c = 0.001$; $k_c = 0.0028$): pressure at controlled nodes (left) and valve SD (right).

of the valve shutter (right – in the logarithmic scale). Each EPS assumes the same control strategy for all *RRTC*-PRVs and, in all cases, valves are assumed to be fully open at the beginning of the simulation. In order to avoid unsteady flow conditions, a maximum displacement $\Delta\alpha = 0.03$ is allowed in $Tc = 5$ min. This constraint causes a delay of about 3 hours in achieving the pressure set-points at *critical* nodes, starting from a fully open valve.

The comparison among all results in Figure 7 hints that the *HL* strategy is expected to provide the best performance in terms of pressure control at *critical* nodes. In fact, the pressure set-point is kept roughly constant in the face of demand variation. *Vice versa*, both *RES* and *SD* (with $k_c = 0.001$) result in similar simulated behaviour, with slightly superior performances for *SD* over *RES*, depending on the PRV–*critical* node considered. In both *RES* and *SD* cases, the most relevant changes in customers' demand (i.e. around 8.00, 12.00, 16.00 and 19.00, see Figure 4) result in difficulties in maintaining a constant set-point at *critical* nodes.

If differential pressure across the PRVs is available, the *HL* strategy proves to be the most effective and stable, besides showing the advantages discussed in Giustolisi *et al.* (2017) and Laucelli *et al.* (2016). If flow measurement is available at PRVs, the *RES* strategy has the advantage of being more flexible in the face of flow (i.e. demand) variations, without requiring the assessment of a constant gain factor. If neither flow nor differential pressure are available, the *SD* strategy is proved to achieve, in this case, similar performance to *RES*. Nonetheless, nothing can be said about the acceptability of the calibrated gain factor in the face of unexpected flow (i.e. demand) variations over time.

For the sake of completeness, Figure 7 also reports the analysis returned by the *SD* strategy, with $k_c = 0.0028$, which results in severe instability in both pressure at the controlled node and valve opening. It can be argued that in a complex system with several *RRTC*-PRVs and variable behaviour of the hydraulic system (e.g. remarkable demand variations over the day), the calibration of k_c is a problematic task. This is because k_c is a dimensional variable depending on the flow rate through the PRV (see Giustolisi *et al.* 2017).

DISCUSSION AND CONCLUSIONS

Technical best practices recommend pressure control via PRVs as a cost effective and reliable approach for leakage reduction in WDNs. *Classic* PRV control schemes are based on achieving a target pressure value just downstream of the valve, thus allowing pressure reading and opening modulation in a single (mechanical) device. *Vice versa*, *RRTC*-PRVs require a more complex apparatus including a remote pressure sensor, communication equipment at the remote site, the communication interface at both ends, PLC units to control the valve and a mechanical actuator, beside the valve itself. Actually, the cost of ICT equipment for *RRTC*-PRVs is quite affordable nowadays and advancements in long-life batteries make it possible to install such devices irrespective of the electric power line availability. In addition, some of the required equipment (e.g. pressure sensors) could be already available in the WDN (e.g. Supervisory Control and Data Acquisition – SCADA systems), thus allowing integration of the *RRTC*-PRV scheme into the existing WDN monitoring and control architecture.

From an operational perspective, the automatic modulation of valve opening based on current WDN hydraulic status makes *RRTC*-PRVs more reliable than *classic* PRVs also, with respect to abnormal scenarios. For example, the abnormal firefighting outflow would cause pressure drop at critical nodes and the automatic opening of the PRVs, without needing additional control actions/procedures on that valve.

All these factors make the *RRTC*-PRVs increasingly appealing over *classic* PRVs.

The decisions among different possible alternatives for implementing *RRTC*-PRVs in real WDNs mainly rely on empirical approaches that are hardly exportable to other similar contexts (e.g. different WDN managed by the same water utility). Moreover, traditional hydraulic models, conceived for WDN design purposes, do not provide adequate support. Indeed, they do not model pressure-dependent *unreported and background (volumetric) leakages* along pipes and are not able to simulate *RRTC*-PRVs controlled by remote *critical* nodes.

This paper, after discussing from a hydraulic perspective the main modelling features to support planning pressure control for leakage reduction, practically demonstrates the application of advanced models.

The case study of North-West Oppegård exemplifies the capabilities of new generation models from both water utilities' and regulatory bodies' perspectives. The possibility of estimating the *unreported and background leakage* (*volumetric real losses*) volume as an indicator of asset deterioration allows support of decisions regarding both required investments (i.e. number of new devices to be purchased and installed) and expected savings in terms of reduction of real losses among alternative scenarios (e.g. Figure 6).

It is worth noting that the calibration of the parameters of the *volumetric leakage* model does not impair the analysis for planning purposes. In fact, although it might affect the absolute value of water loss volume, it still allows comparison among alternative pressure control scenarios in terms of leakage reduction. This means that, although the accuracy of the hydraulic model can be progressively increased (e.g. by monitoring flow/pressure though the system), it effectively assists in taking effective decisions.

The Oppegård case study shows that the ILI is not consistent in assessing leakage reduction achievements. In more detail, the analysis reported herein shows that, depending on the current leakage rate and pressure control scheme, the ILI might be invariant or even increase in the face of a large reduction of leakage volume from the controlled network. This happens because a single lumped index (i.e. ILI) is not able to reproduce the combined effect of pressure reduction along each pipe in the WDN. In fact, it might happen that two scenarios showing about the same average network pressure have different water loss volumes.

The Oppegård network is also used to demonstrate the effectiveness of advanced WDN hydraulic models to support system operation, comparing three alternative strategies for the electric real-time regulation of *RRTC*-PRVs. Although careful field tests are recommended to implement new pressure control strategies, the analysis reported herein provides some useful support. In particular, it shows that the *HL* strategy overcomes the other strategies (*RES* and *SD*), at the cost of differential pressure measurements across the PRVs. The *RES* strategy shows here a similar control performance to *SD* (with $k_c = 0.001$), although the additional flow measurements at PRVs are expected to increase robustness in the face of unexpected changes in valve flow.

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The original data used in the Oppegård case study are confidential and the authors do not have permission to share them with other people.

The case studies reported herein have been accomplished by means of the WDNetXL Pressure Control Module, which can be requested free of charge for students and research purposes at www.idea-rt.com. The data used in WDNetXL can be obtained by contacting Prof. Orazio Giustolisi (*orazio.giustolisi@poliba.it*).

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