Performance Assessment of Water Distribution Systems Subject to Leakage and Temporal Variability of Water Demand

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Abstract

A water distribution network (WDN) is designed and managed to provide a reliable water supply, that is to properly guarantee the water request by users, particularly in critical operating conditions such as those of peak demand. Therefore, the assessment of the influence of the water demand characteristics is an essential requirement in the context of the WDN reliability. In this paper the impact of the pattern of hourly demand on the WDN performance is analysed for a system subject to aging processes and the pipe temporary unavailability, and also affected by water losses with different leakage levels. The hydraulic deficit which can occur when the pressure falls below the minimum service value is assumed as performance index, and its relevance is analysed without and with preventive maintenance. The case of the synthetic Anytown network is analysed, but the procedure is of general validity and can be applied to any real WDS. Defined in a prescribed temporal horizon the pipe replacement prioritization without preventive maintenance, the effects of pipe substitutions are analysed as a function of different scheduling times to quantify the reduction of the hydraulic deficit. The results show the capability of the proposed approach to define a pipe replacement prioritization and the related scheduling time, in view of the relevance that these aspects could have in any economic analysis developed to define a proper maintenance strategy.

Keywords: water distribution system performance; leakages, variable water demand; pipe replacement prioritization

Introduction

Water distribution networks are essential infrastructures for communities and it is of fundamental importance that they continue to operate efficiently and economically within defined operating requirements and over an extended period (Engelhardt et al. 2000). The degradation due to ageing of the networks reduces their mechanical and hydraulic characteristics, making the pipes subject to malfunctions and failures with increasing frequency and causing a general lowering of the piezometric surface over time. As a consequence, the level of service reduces in terms of both quantity and quality and only an adequate rehabilitation strategy may restore the regulatory requirements. The costs associated with such network maintenance operations are relevant, and the decision on interventions must be taken on the basis of technical and economic considerations over an extended period.

Different researchers have addressed specific performance metrics within a rehabilitation plan such as maximizing reliability and resiliency, as well as minimizing leakage (e.g. Araujo, Ramos, and Coelho 2006) and failure risk (e.g. Giustolisi, Laucelli, and Savic 2006). In addition, the strategy should aim to increase the economic efficiency of the water company in operating its distribution networks. Numerous researchers employed multiobjective optimization models, especially multiobjective genetic algorithms, for solving WDN rehabilitation problems (e.g. Nafi and Kleiner 2009). Among the different objectives considered, there are generally the powering and maintenance of a highly reliable water supply, while minimizing the total cost of operation (Dandy and Engelhardt, 2006; Alvisi and Franchini, 2009).

WDN reliability is generally assessed through performance indicators that relate the water delivered to users to their demands under critical operational scenarios due to either mechanical or hydraulic failure. Therefore, in the evaluation of the system performance during a prescribed time horizon, all the factors affecting its continued reliability should be considered, such as the ageing of pipes or components, and the temporary unavailability of some components, e.g. pipes or pumps (e.g. Kleiner et al., 1998; Mazumder et al., 2019). During these extended period analyses the pressure head may often prove to be insufficient and that is many cases of pressure deficit may occur. To correctly reproduce the condition of insufficient nodal head the hydraulic simulations should be carried out by a pressure-driven solution (e.g. Giustolisi et al., 2008).

In the assessment of the WDN performance the description of the demand by users is an aspect of absolute importance and the uncertainty in nodal demands and their variation with time is one of the main sources of error in the WDN analysis. The temporal variability of water demand is characterized by a long-term trend, depending on the socio-economic development of the territory, a seasonal trend, linked to the climatic aspects and to economic factors such as tourism, and a daily trend, linked to user habits. Since the long-term trend, as well as the seasonal trend, is controlled or mitigated by the appropriate management of supply and storage reservoirs, only the daily variation is usually taken into account when calculating the overall performance of a WDN; sometimes a random component is introduced with an assigned probability distribution to represent the fluctuations of the demand at finer temporal scale (e.g. Creaco et al., 2018). The spatial variability of the request may be neglected if the network skeletonization ensures a proper lumping of the spatial request and the latter is distributed among different nodes according to the type of user, keeping the ratio between the demand in each node and the total consumption constant. However, a residual variability of the spatial distribution of the demand can be taken into account by assuming on each node a stochastic fluctuation independent from the other nodes (e.g. Darvini, Salandin and Da Deppo 2009).

Among factors connected with the water demand behaviour, temperature is the most relevant because it directly influences several sources of water consumption such as showers or water for gardens. Water consumers also respond to the occurrence of rainfall and other climate variables, though rainfall seems to have a dynamic effect, in the sense that it reduces water demand initially, but the effect diminishes over time (e.g. Herrera et al. 2010).

In practical applications, the simulation of the residential water demand is often carried out by assuming averaged values, both in space and in time, of water demands. As previously stated, the spatial averaged values are obtained by clustering the water consumption of users at each node of the network, while the time averaged values are obtained as the mean of the instantaneous values of the nodal demands. The simultaneous use of both simplifications could not be considered much reliable for the hydraulically disadvantaged zones of the network.

The present work analyses the influence of the temporal variability of water demand on the evaluation of the performance of a WDN subject to mechanical failure and water losses, considering different leakage levels with leak positions uniformly distributed on space.

The proposed method is based on a probabilistic approach (e.g. Wagner et al. 1988) able to take into account the processes of hydraulic and mechanical deterioration of the elements that make up the system and the cases of insufficient pressure that can occur in long-term simulations, as in the case of breakage of one or more pipes. The hydraulic model of pressure-driven type allows to accurately model the nodal losses and to calculate the volume undelivered to users when the nodal pressure falls below the service level. This undelivered volume, computed in the whole system or related to the single pipe unavailability, is assumed as performance index.

The average annual consumption of the entire system is distributed on each node according to the user type and kept constant throughout the temporal horizon considered. The variability of demand over time is here simulated by introducing an hourly pattern of the nodal flows respect to the average value delivered in the year, but the suggested method could easily be applied to networks affected by a long-term trend and/or seasonal fluctuations.

The paper is organized in different sections describing: a) the general formulation of the extended period simulations, where details on the structural aging of pipes are given together with a thorough description of the Monte Carlo procedure adopted to develop the probabilistic approach; b) a resume of the leakage model adopted; and c) the formulation of the performance index based on the volumetric deficit. The proposed approach of general validity is then applied to the synthetic Anytown network (Darvini, 2014), to evaluate its performances in a specific case.

The results discussed and the conclusions presented at the end of the paper show that the assumption of variable demand over time leads to volumes undelivered to users generally larger than in the case of demand assumed constant and equal to the annual average. Moreover, for the specific WDN examined, the reduction of the undelivered volume consequent to the substitution of some pipes is evaluated for different intervention times.

Assessment of reliability in WDSs subject to leakage and temporal variability of the user demand

Probabilistic approach in the Extended Period Simulations (EPS)

The proposed approach for the reliability assessment is based on the use of Monte Carlo simulations. The time of failure T_f and the time T_r for the system to return to operation are random variables with assigned probability density function (pdf) $f(T_f)$ and $g(T_r)$ respectively. From the knowledge of $f(T_f)$ and $g(T_r)$ of each network component, a sample life cycle of the system could be reproduced as a succession of normal and failure states of all the elements. This succession constitutes a single Monte Carlo simulation (MCS).

The MC procedure allows for the analysis of the aspects related to the mechanical failure of pipes, also considering their ageing. Different pdfs of failure and repair times could be assumed, and the influence of the choice of the probability distributions on the reliability assessment was evaluated in Darvini (2014). Under the assumption of an exponential pdf the mean time to failure is MTTF = $1/\lambda$, where the failure rate λ is commonly assumed constant. Since the pipes deteriorate, the failure rate increases in time according to the exponential law given by Shamir and Howard (1979)

$$\lambda(D,t) = \lambda(D,t_0)\exp[A(t-t_0)] \quad (1)$$

where $\lambda(D,t)$ is the failure rate of the pipe of diameter D at the generic time t, t_0 is the time of installation of the pipe, and A is the coefficient of breakage rate growth (yr^{-1}). The initial value $\lambda(D, t_0)=1/MTTF(D, t_0)$ could be deduced for each pipe from the existing relationships that relate the failure rate and the pipe diameter, the latter being recognized as one relevant characteristic that among others affects the breakage rate (e.g. Pelletier et al., 2003).

Defined as ε_n (m) the initial roughness of the pipe when it was new ($t=t_0$) and ε_o (m) the constant roughness characterizing the aged pipe, the pipe roughness behaviour during time is

$$\varepsilon(t) = \varepsilon_o - \exp(\beta t)(\varepsilon_o - \varepsilon_n) \quad (2)$$

where β is roughness growth rate (m yr^{-1}). Instead of the linear relationship usually used to model the effect of aging on the carrying capacity of pipes (Sharp and Walski, 1988; Kleiner et al., 1998), the exponential equation (2) is of more general form and it permits to bound the roughness value in a prefixed range.

Each MCS consists of three steps. 1) At the beginning of the simulation, all the system pipes are normally operating. For each pipe the time to failure T_f is generated based upon the assumed exponential probability distribution at the time t. Among all the pipe of the system, the one with the shortest time to failure is considered out of work, and the current time is increased of T_f. 2) The hydraulic simulation is run and all the computed hydraulic quantities required for the reliability evaluation are stored. 3) Due to the deterioration process of pipes, the failure rate λ increases with time, and a new generation is needed. A time to repair T_r is generated for the out of work pipe, whereas for the other pipes a new value of time to failure T_f is generated. If the T_r of the failed pipe is longer than the smaller T_f of the other operating pipes, the number of the unavailable pipes increases by one unit. Otherwise, all the components will result normally operating. Accordingly, the current time is increased of the minimum among the T_r and all the T_f. Steps 1-3 are repeated until the end of the planned horizon, then the quantities needed to compute performance index are evaluated and stored. When the number of realizations reaches the given maximum value, the statistics on the stored values are calculated to provide the system performance estimate.

The hydraulic analysis is developed using a numerical algorithm that integrates the simulator by Todini and Pilati (1988) and the iterative procedure by Todini (2003) to properly solve the cases of insufficient head that may occur under some operating conditions without specific assumption on the relationship between the nodal flow rate and pressure.

Leakage model

In the pressure-driven solution the outflows Q_i at the unknown head nodes are assessed as a function of the nodal demand and pressure head, and leakage can be accurately modelled. The nodal flow rate Q_i can be calculated as the sum of the water delivered to the users Qs_i and the leakage allocated to the nodes Ql_i.

The leakage model focuses on representing the background losses resulting from leaks that are neither detectable nor locatable, and the undetectable leaks which are however locatable through leakage detection campaigns. These types of leaks are of low intensity but persist over time without being identified, thus resulting in high volumes of water losses. On the contrary, detectable leaks are associated with breaks in medium or large-sized pipes and they are generally located and eliminated in a short time by repairing the damaged pipe. Therefore, though they may be characterized by high flow losses, their brief duration means that relatively small volumes will be lost overall (Alvisi and Franchini, 2009). Despite this fact, due to the failure of one or more pipes, a pressure shortfall may affect a portion of the WDN, and in that area the nodal demand may not be met along all the time needed to repair the damaged elements.

The background losses and the undetectable leaks in WDS were modelled relying on the model proposed by Germanopoulos (1985):

$$Ql_i = Cl_i H_i^{N1_i} \tag{3}$$

where Ql_i is the leakage flow rate at i-th node, H_i is the pressure head at i-th node, Cl_i is the leakage model coefficient and $N1_i$ is the leakage model exponent.

Values of leakage exponent ranging from 0.5 to 2.79 have been reported from experiments and field studies (e.g. Cassa and Van Zyl 2014). Factors that cause this variation on the leakage exponent include pipe material, leak hydraulics, soil hydraulics and water demand.

Reliability Assessment

The hydraulic failure of the WDN occurs when the water supplied to users is inadequate in comparison with the demand. Thus a hydraulic failure is deemed to occur during all the time period T_k when at node i the supplied flow $Qs_{i,k}$ is smaller than the nodal demand $Qd_{i,k}$.

When a demand pattern is taken into account, both the nodal demand $Qd_{i,k}$ and the supplied flow $Qs_{i,k}$ are variable in time. The nodal undelivered volume may be computed as:

$$Vu_{i,k}(T_k) = \left[Qd_{i,k}(T_k) - Qs_{i,k}(T_k)\right] T_k \tag{4}$$

In an extended period simulation, a single value can be calculated at each network operating instant. Therefore, the characterization of the system during the planning horizon at each MCS can be carried out by calculating the total undelivered volume to users as:

$$Vu(mc) = \sum_{i=1}^{N} \sum_{k=1}^{K} \left[Qd_{i,k}(T_k) - Qs_{i,k}(T_k)\right] T_k \tag{5}$$

where N is the number of supply nodes of the system and K is the number of time periods in which the analysed lifespam is subdivided.

From the results obtained for each MCS, the expected value of total undelivered volume to users is given by:

$$\mathrm{EVu} = \frac{1}{\mathrm{NMC}} \sum_{mc=1}^{NMC} \mathrm{Vu}\,(mc) \qquad (6)$$

where NMC is the maximum value of MC runs.

Through the Monte Carlo analysis it is possible also to evaluate EVu_j, that is the contribute given by the j-th pipe to the total undelivered volume EVu.

Case study

The analysis was applied for the synthetic water distribution network of Anytown modified as in Darvini (2014). In the examples the network scheme is simplified as shown in Figure 1. Node elevations and mean nodal demands are reported in Table 1, pipe data and initial roughness are shown in Table 2. The mean time to repair MTTR is set constant for all the pipes and time invariant, while the MTTF=1/λ reduces in time and it is a function of the pipe diameter, as previously stated in eq. (1).

The pdf of the time to repair is deduced from the analysis of measured data in the WDN of Marghera (VE), in Italy (Salandin, 2003). The time to repair distribution is described by a log-normal probability distribution: mean and variance of the natural logarithm of the time to repair are 2.93 ln(hours) and 0.362 $(\ln(\mathrm{hours}))^2$ respectively. On the basis of the available information about pipe breaks occurred in the Marghera network, it was possible to recognize the following relationship between MTTF (yr) and the diameter of pipes

$$\mathrm{MTTF}(D_j) = \left[0.2688\exp\left(-0.0023 D_j\right)\right]^{-1} \qquad (7)$$

where Dj is the diameter of the j-th pipe, given in millimeters. Eq. (7) was assumed to define the MTTF in our synthetic network at the initial time $t=t_0$, while the mechanical decay coefficient A of eq. (1) is set to 0.1 yr^{-1}. To avoid the hydraulic failure of the entire distribution system, a large MTTF is artificially assigned to pipe 4.

To describe the increase of roughness in time, in eq. (2) is assumed $b=0.15\ yr^{-1}$. This leads to pipe aging in 20-30 years according to Sharp and Walsky (1988). The initial values ε_n are reported in Table 2, while the final values ε_o are obtained doubling the initial ones.

The illustrative example has been developed by considering the fulfilment of the flow service value with reference to the service piezometric head Hs_i = 25 m on each node. The total head of the reservoir located at node 20 is assumed constant and equal to 82 m a.s.l. The period of simulation was set equal to 30 years, and 500 MCS were developed to ensure the convergence of the required statistics. This duration may be shorter than the actual lifetime of a WDN which in several cases is longer than 50-70 years, but may be an adequate planning horizon for the system management (e.g. Roshani and Filion 2014; Mala-Jetmarova, Sultanova and Savic 2018).

The effects on the system performance of three leakage levels (13%, 25%, 50%) are analysed by considering a single leakage coefficient Cl_i for all the nodes and $N1_i$=1.0 (eq. 3), being the leakage percentages chosen according to recent reports on the leakage behaviour in Italy (ISTAT 2018). In the following examples the mean annual water demand as well as the Cl_i and $N1_i$ coefficients are assumed constant during the planning horizon. The temporal variation of the user demand is represented by the three hourly patterns p1, p2, p3 illustrated in Figure 2 (Milano 2012). The pattern p1 is characterized by a large discrepancy between the water demand at each hour during the day and the constant daily demand, thus showing higher hourly coefficients. These hourly coefficients vary between 0.2 and 1.56 within 24 hours, the minimum occurs between 2 and 3 a.m. and the maximum at 12 p.m. The pattern p2 presents reduced fluctuations respect to p1, varying the hourly coefficients between 0.4 and 1.38. The pattern p3 is closer to the constant pattern p0, with limited fluctuations bounded in the range 0.52-1.32.

Discussion on the results

Influence of the leakage level on the hydraulic deficit

The expected value EVu of the volume undelivered to users during the planning horizon volume is illustrated in Figure 3 as a function of the level of leakage for the pattern p0, p1, p2, and p3 considered. When pipes are broken the nodal pressure could be insufficient to satisfy the request of the users and a fraction only of the water demand can actually be delivered. We stress that EVu is a performance index affected by leakage, but it does not account for the volume of leakage. In other words, an identification of the leakage positions (e.g. Ruzza et al., 2015) is outside of the aim of the present work, and the pipes leading to the greatest EVu values are not necessarily affected by maximum leakage.

From a general point of view, the influence of the pattern is more significant for smaller leakage levels than for a large percentage of leakage. For a leakage level of 5% the expected value of the undelivered volume increases of about 600% from the pattern p0 to p1, while for a leakage level of 50% EVu increases of about 100%. The hydraulic deficit is at the minimum in the absence of leakages and increases with the leakage level in non-linear manner. For a leakage level from 5% to 50% the undelivered volume increases from $1x10^4$ m^3 to $8x10^4$ m^3 for a constant pattern, while for p1 the undelivered volume increases from $7x10^4$ m^3 to $16x10^4$ m^3.

Moreover the rate of increase of Evu increases moving from p3 to p1, while for a constant pattern its behaviour is more complex. Up to percentage of leakage of 20-25% the rate is less than the case p3, but it grows up rapidly and for the 40-45% it results larger than the one in case p1.

This general result can be explained considering that in the case p0 the water demand is constant and equal to the mean daily value, so that the mechanical unavailability affects the nodal pressure distribution regardless of break instant. If a pattern exists, the out of order of one or more pipes causes a more relevant pressure shortfall when the request is higher than the mean value, and this due to the non-linearity of head losses with the discharge in pipes. In other words, the EVu reduction corresponding to the low water demand, does not compensate its increase during high request time intervals.

By considering the behaviour of each pipe over the entire planning horizon, the expected value of the number of breaks computed over 500 MCS is reported in Figure 4. According to equation (7) the conduits which present the maximum number of breaks are pipes 24 and 36, the failure rate being much higher for smaller diameters. These pipes present a mean value of breaks equal to 18 times over 30 years, while the other pipes break less frequently. As previously stated, due to the large MTTF assigned to pipe 4, it doesn't present any break and the hydraulic failure of the entire distribution system is avoided.

In the planning horizon considered, pipes 2, 3 and 22 show less than 5 breaks, eight pipes (1, 7, 10,12, 20, 25, 38 and 39) have more than 5 but less than 10 breaks, while all the remaining pipes show more than 10 but less than 20 breaks.

Thanks to the system redundancy, the mechanical unavailability does not lead necessarily to hydraulic failure, the latter one occurring when the nodal pressure is lower than the service level and thus the nodal demand cannot be fully accomplished. The expected value of the undelivered volume (m^3) due to the breaks of each pipe in the planning horizon of the system EVu_j is illustrated in Figure 5 for different leakage levels in the case of both constant pattern p0 and time-variable pattern p1. The comparison between the undelivered volume associated to the mechanical failure of every single pipe and the mean number of the breaks occurred on that pipe in the planning horizon shows that the hydraulic deficit EVu_j is not proportional to the number of j-th pipe breaks. In fact, the Anytown network is a redundant system and in case of unavailability of one or more pipes the water follows different paths to reach anyway the supply nodes.

For a constant pattern, Figure 5a shows that in the absence of leakage, the hydraulic failure, and thus a positive value of EVu_j, is given only when pipes 2, 3, and 38 are subject to mechanical unavailability. In other words, the pipes having larger diameters and thus larger

MTTF present smaller number of breaks, but their mechanical unavailability produces, as expected, the most serious conditions of hydraulic deficit.

As the leakage level increases the number of pipes that lead to hydraulic failure increases. For a leakage level of 50% seven pipes give undelivered volume to users when they are broken, that is pipe 2, 3, 16, 20, 22, 34 and 38. As the leakage percentage increases, also the undelivered volume due to the breaks of each pipe increases for all the conducts.

Assuming the same leakage level, when the user demand is variable in time according to pattern p1, the number of pipes producing hydraulic failure with their breaks increases respect to the case p0, with a corresponding increase of the undelivered volume. In the case of no leakage, besides pipes 2, 3 and 38 that are responsible for undelivered volume also in the case p0, the unavailability of pipes 16, 20, 22, 25 and 34 causes undelivered volume to users. For a leakage level of 50 % the pipes responsible for undelivered volume become twelve. The value of EVu_j increases for all the non-functioning pipes and thus the total undelivered volume of the system increases, according to the results of Figure 3. For the maximum leakage level (50%), in both cases p0 and p1 the pipe which contributes mainly to the hydraulic failure of the system is the number 20, while its mechanical failure does not give hydraulic failure when the pattern is constant and the leakage level is smaller than 25%.

To assess the intervention priorities among all the pipes subject to failure and help the utilities in the management of the distribution systems, the contribution of each pipe to the inability of the system to satisfy the user request in terms of delivered water is analysed. Figure 6 shows the percentage ratio between the expected value EVu_j of the undelivered volume consequent to the breaks of every single pipe and the expected value of the total undelivered volume of the system EVu.

For a constant demand pattern and no leakage three pipes concur to the hydraulic deficit of the system and pipe 38 is responsible for more than 50% of the total undelivered volume. When the leakage level is 13%, the pipes which contribute for more than 60% to the total undelivered volume of the system are number 2 and 38. Also pipes 3 and 16 significantly concur to the hydraulic deficit. The pipe giving the smallest contribution is number 34. As the leakage level increases the number of the pipes concurring to EVu increases, and the impact of each single pipe on the total hydraulic deficit reduces. The contribution of pipes 2, 3 and 38 to the total deficit reduces and tends to become similar to others. For instance, for a leakage level of 50% the contribution of pipe 3 is smaller than that of pipe 34.

In term of percentage, similar results are obtained for the pattern p1 (Figure 6b), although the impact of the leakage level is less appreciable. For the maximum leakage level, thirteen pipes have a role in the total hydraulic failure and the contribution of every pipe on the total undelivered volume become evenly more distributed among all the elements.

This means that, as EVu is assumed as performance index, the leakage percentage can affect the planning of a WDN rehabilitation. For smaller leakage level, the greatest benefits in terms of reduction of the overall undelivered volume can be obtained by replacing the pipes which give the maximum contribution to the total undelivered volume, despite the fact that they break rarely. In the case of larger leakage percentages, the same benefits can be obtained by replacing the pipes characterized by a smaller diameter and a large number of breaks.

Influence of the demand pattern on the hydraulic deficit

The influence of the hourly pattern of the water demand is illustrated in Figure 7, where the EVu_j is shown for different demand patterns and in the leakage level of 50%, besides the no leakage case.

For almost all the pipes the variability of the user demand during the day produces an increase of the undelivered volume moving from pattern p0 towards p1.

When the water demand is constant during the day (pattern p0), some pipes don’t produce hydraulic deficit as a consequence of their mechanical unavailability, but when an hourly variability is introduced in the demand pattern this is no longer true. Some pipes, like pipe 25 in the no leakage case, lead to hydraulic deficit in the system only for the extremely variable pattern p1.

For the no leakage case, the pipe mostly responsible for the hydraulic deficit depends on the shape of the pattern: pipe 38 for p0, pipe 2 for p3 and pipe 20 for p2 and p1. On the contrary, for a leakage level of 50% the pipe mostly responsible for the hydraulic deficit is always pipe 20. This means that for the more variable pattern p2 and p1 the pipe mostly responsible for the hydraulic deficit does not depend on the leakage.

The influence of the pattern on EVu_j varies with the leakage. In the no leakage case, the increase of EVu_j according to the pattern shape is enhanced compared to larger leakage percentage. To give an example, for pipe 20, moving from p0 to p3 EVu_j varies from 0 to 6827 m^3, while from p3 to p2 the increase is around 50% and from p2 to p1 it is 47%. For a leakage percentage of 50% the variation of EVu_j with the pattern is smoothed, since all failed pipes are responsible for hydraulic deficit for the constant time pattern p0 also. For the pipe 20, the EVu_j variation is about 33% moving from p0 to p3, 12% from p3 to p2 and 14% from p2 to p1.

Reduction of the hydraulic deficit by pipe replacement

As previously shown, for a limited leakage percentage (13%), only few pipes, the ones characterized by larger D_j, significantly concur to the hydraulic deficit although their number of breaks is small. When the leakage level increases, the number of pipes concurring to the undelivered volume increases, and this means that the contribution of single pipes to the total deficit reduces and tends to become similar to the one of other pipes. This general result is manifest in the developed example from Figure 6.

The reduction of the total undelivered volume of the system obtained with the replacement of some pipes, one at a time, is illustrated in Figure 8 for different years of intervention and for a leakage level of 13% and 50%. The results of Figure 8 are obtained for the constant mean daily demand pattern p0.

For a leakage level of 13% the maximum benefit (~24%) in terms of reduction of the undelivered volume is obtained by substituting the pipe 2 (Figure 8a). The replacement of the pipe 38 also leads to a significative reduction of the hydraulic deficit, of the order of 20% (Figure 8b). A minor advantage is obtained by the substitution of pipe 16 (Figure 8c), while small benefits are gained from the substitution of pipe 20 (Figure 8d). Otherwise the replacement of pipe 20 gives just a reduction of the pipe roughness, but this impact on the hydraulic deficit is negligible with advantages in term of EVu reduction s smaller than 5%.

When the leakage level is 50% the benefit from the replacement of pipe 2 reduces to 14%, and the replacement of pipe 38 also gives improvements smaller than the ones computed for leakage percentage of 13%, reducing the maximum benefit to 8%. The impact of the substitution of pipe 16 does not vary with the leakage percentage. The replacement of pipe 20 seems to give the largest benefit, having a maximum reduction of about 17% in terms of undelivered volume.

These results show that the leakage percentage can affect the planning of a WDN rehabilitation, although its impact is different for different pipes. For smaller leakage levels, the greatest benefits in terms of reduction of the overall volume undelivered to users are obtained by replacing the pipes which give the maximum contribution to the total volume EVu, even if they break rarely. In the case of larger leakage percentages, the same benefits can be also obtained by replacing pipes characterized by a smaller diameter and a larger number of breaks. This opportunity may lead to minor replacement costs and reduces the disruptions of supply to users.

The reduction of the undelivered volume to users obtained by replacing the same pipes in the case of the pattern p1 are reported in Figure 9. As expected, the substitution of the pipe gives a smaller benefit in terms of reduction of the undelivered volume to users in comparison with the case of constant demand pattern. Thus, the replacement of a specific pipe does not produce a benefit larger than that of a different substitution. This is evident for the case of a leakage level of 13% where the replacement of the pipes 2 and 38 produces a benefit between 8% and 12%, significantly smaller than the corresponding benefit obtained for the pattern p0. For the pipe 16 also the benefit obtained by the substitution reduces as the pattern becomes variable in time. On the contrary, for the pipe 20 the advantage gained with the substitution significantly increases when the pattern changes from p0 to p1. Thus, for a leakage percentage of 13%, the lager hydraulic benefit is reached with the replacement of the pipe 20, while in the case of the pattern p0 the break of the same pipe doesn't produce any hydraulic deficit and the only advantage of its substitution is related to the reduction of the pipe roughness. When the leakage level increases up to 50% the advantage of the pipe replacement decreases respect to the case of smaller leakage percentage and the differences between two cases are not relevant.

Therefore, for a largely variable demand pattern i) several pipes may contribute to hydraulic deficit of the system when they break, ii) there are no pipelines to be replaced with absolute priority and iii) the scheduling of pipe substitution has to be defined on the basis of additional criteria. This result seems to be scarcely affected by the leakage level.

Finally, observing the behaviour of the total undelivered volume reduction according to the replacement time it is possible to schedule the optimal year of intervention for improving the system performance. For the synthetic WDN analysed, in all the cases reported here, the optimal year to replace pipes is between 15 and 20 years. If the substitution is made earlier the pipe ages further during the WDN lifetime, while a later intervention does not produce significant advantages. Obviously, an economic analysis should be applied to integrate these results with cost-effective criteria.

Conclusions

The influence of the temporal variability of water demand on the evaluation of the performance of a WDN subject to uniformly distributed leak positions is analysed by considering different leakage levels of the system. Four demand patterns have been analysed varying from a constant behaviour corresponding to the mean daily value to a variable one characterised by an hourly peak coefficient of 1.56. The WDN reliability is evaluated in terms of volume undelivered to users (EVu) due to the insufficient value of nodal pressure respect to the minimum service level needed to fully supply the water request by users. The head driven analysis is carried out by taking into account both the mechanical unavailability of the pipes and their aging process.

The results show that i) the expected value of EVu during the planning horizon depends on the characteristics of the demand pattern and on the level of leakage, and ii) the hydraulic deficit increases in a non-linear manner with the leakage percentage. The rate of increase with the leakage percentage is larger as the hourly pattern departs from the constant mean daily value, being in the latter (limit) case of constant demand the rate of increase different respect to other variable patterns and depending on percentage of leakage. In term of total amount, the EVu grows moving from a constant water demand to variable patterns and as the leakage increases, but the relative impact of the pattern behaviour is more significant for smaller leakage levels.

The contribution of each pipe on the EVu is also considered. For a constant pattern only few pipes, but among the largest in terms of diameters, concur to the whole hydraulic deficit, though they are affected by a limited number of breaks. Thus, the substitution of these pipes provides a large benefit in terms of reduction of EVu, and this occurs mostly when the leakage percentage is limited. As the leakage level increases, the number of pipes that - due to the mechanical failure - yield hydraulic deficit increases, and equivalent advantages in term of reduction of EVu can be obtained by replacing also pipes characterized by small diameters and a larger number of breaks. This is more manifest with patterns highly variable in time when, in case of breakage during the periods of maximum demand, not only largest pipes are responsible for situations of hydraulic deficit, and no conduit plays a fundamental role with respect to the others. Thus, the priority in the replacement can be assigned also to pipes characterized by smaller diameter and large number of breaks, with the advantage to reduce the inconvenience for users related to the number of repeated repairs.

In conclusion, the leakage percentage affects the planning of the pipe substitution mainly in the WDNs characterized by a demand pattern with limited variations during the time. In such cases it seems appropriate managing the network with the replace of pipes with limited mechanical unavailability. Obviously the assessment of scheduled pipe replacement deduced from the analysis of the undelivered volume to users EVu must be integrated with an economic analysis not considered here because outside the aim of this work. Introducing costs of pipe repair/replacement for any specific real world WDN, the pipe replacement prioritization deduced from the suggested method could be modified, especially dealing with long conduits of large diameter. Future developments of this study will consider the influence on the WDN reliability of the variability of the user demand.

Data availability

All data, models, or code generated or used during the study are available from the corresponding author by request.

Notation

The following symbols are used in this paper:

A = coefficient of breakage rate growth

Cl_i = leakage model coefficient at i-th node

D_j = diameter of the j-th pipe

EVu = expected value of total undelivered volume

EVu_j = expected value of total undelivered volume due to the breaks of j-th pipe

$f(T_f)$ = probability density function of the time to failure

$g(T_r)$ = probability density function of the time to repair

H_i = pressure head at i-th node

Hs_i = service pressure heads at i-th node

K = total number of states

MCS = simulation Monte Carlo

MTTF = mean time to failure

MTTR = mean time to repair

N = total number of nodes

$N1_i$ = leakage model exponent at i-th node

NMC = maximum number of Monte Carlo simulations

Q_i = flow rate at i-th node

$Qd_{i,k}$ = water demand at i-th node in the k-th state

Ql_i = leakage flow rate at i-th node

Qs_i = flow rate delivered to users at i-th node

$Qs_{i,k}$ = flow rate delivered to users at i-th node in the k-th state

t = current time

t_0 = initial time

T_f = time to failure

T_k = duration of k-th state of hydraulic failure

T_r = time to repair

Vu(mc) = total undelivered volume in the mc-th Monte Carlo simulation

$Vu_{i,k}$ = undelivered volume at i-th node in the k-th state

β = roughness growth rate

ε = pipe roughness

ε_n = initial roughness of the pipe

ε_o = roughness of the aged pipe

λ = failure rate

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Table 1. Node elevation and nodal demand data for the illustrative network.

Node	Elevation (m a.s.l.)	Demand (l/s)	Node	Elevation (m a.s.l.)	Demand (l/s)
1	6	31.5	11	36	25.2
2	15	12.6	12	15	31.5
3	15	12.6	13	15	31.5
4	15	37.9	14	15	31.5
5	24	37.9	15	15	31.5
6	24	37.9	16	36	25.2
7	24	37.9	17	36	63.1
8	24	25.2	18	15	31.5
9	36	25.2	19	15	63.1
10	36	25.2	20	6	0.0

Table 2. Pipe data for the illustrative network. For each pipe the initial node N1, the final node N2, the diameter D, the length L, the initial values of roughness e and MTTF are reported.

pipe	N1	N2	L (m)	D (mm)	e (mm)	MTTF (yr)	pipe	N1	N2	L (m)	D (mm)	e (mm)	MTTF (yr)
1	1	2	3657.6	304.7	1.5	7.50	21	12	18	1828.8	355.6	2.0	8.43
2	1	12	3657.6	762.9	2.0	21.51	22	13	14	1828.8	762.2	2.0	21.47
3	1	13	3657.6	699.6	2.0	18.59	23	13	18	1828.8	304.7	2.0	7.50
4	1	20	30.5	457.2	1.0	10.65	24	13	19	1828.8	152.4	2.0	5.28
5	2	3	1828.8	253.9	1.5	6.67	25	14	15	1828.8	598.0	2.0	14.72
6	2	4	2743.2	203.9	1.5	5.95	26	14	19	1828.8	253.9	2.0	6.67
7	2	13	2743.2	304.7	2.0	7.50	27	15	16	1828.8	253.9	2.0	6.67
8	2	14	1828.8	253.9	1.5	6.67	28	15	19	1828.8	253.9	2.0	6.67
9	3	4	1828.8	253.9	1.5	6.67	29	16	17	1828.8	203.1	1.5	5.94
10	4	8	3657.6	203.1	1.5	5.94	30	16	18	1828.8	203.1	2.0	5.94
11	4	15	1828.8	253.9	1.5	6.67	31	16	19	1828.8	253.9	2.0	6.67
12	8	9	3657.6	203.1	1.5	5.94	32	17	18	1828.8	203.1	1.5	5.94
13	8	15	1828.8	253.9	1.5	6.67	33	18	19	1828.8	253.9	2.0	6.67
14	8	16	1828.8	203.1	1.5	5.94	34	4	5	1828.8	355.6	1.0	8.43
15	8	17	1828.8	203.1	1.5	5.94	35	5	6	1828.8	406.4	1.0	9.47
16	9	10	1828.8	304.9	1.5	7.50	36	6	7	1828.8	152.4	1.0	5.28
17	10	11	1828.8	394.6	1.5	9.22	37	6	8	1828.8	203.9	1.0	5.95
18	10	17	1828.8	355.6	1.5	8.43	38	7	8	1828.8	609.6	1.0	15.12
19	11	12	1828.8	203.1	1.5	5.94	39	11	17	2743.2	406.4	1.0	9.47
20	12	17	1828.8	606.9	1.5	15.02							