

A rehabilitation decision-making approach for WDNs based on extended period simulations

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ABSTRACT

The paper addresses the relevant topic of the rehabilitation in water distribution networks (WDNs). The degradation due to ageing of the networks reduces their mechanical and hydraulic characteristics, making the pipes subject to malfunctions and failures as well as to the increase of the leakage amount. To ensure a proper service in term of quantity and quality of the water delivered to users, a suitable maintenance program must be implemented, but the associated costs are relevant, and the interventions must be planned on the basis of both technical and economic constraints. This subject is here tackled by a Monte Carlo (MC) approach, considering both the hydraulic and mechanical deterioration processes of pipes affected also by leakage, as usual in the real world. Within the MC iterations, first a probabilistic model simulates the chain of repair and failure conditions, and then the performances of the each WDN configuration is evaluated by use of an efficient pressure-driven hydraulic algorithm. The analysis of the synthetic Anytown network leads to some unexpected results in terms of pipe replacement prioritizing, and due to a different leakage percentage affecting the network, the order of priority in the pipe maintenance program may result substantially modified.

Keywords: water distribution system rehabilitation, leakage, pipe prioritizing

1 INTRODUCTION

A rehabilitation decision model ensures that a water distribution system (WDN) continues to operate efficiently and economically within defined operating requirements over an extended period corresponding to the lifetime of the work [1]. 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 level over time. As a consequence, the level of service reduces in terms of both quantity and quality and only a rehabilitation strategy may restore the regulatory requirements. The costs associated with such network maintenance are relevant, and the decision on interventions must be taken on the basis of technical and economic considerations over an extended period. Numerous rehabilitation measures and decision models have been reported in the literature, but there exists no universal agreement on how performance should be measured in a rehabilitation decision model [1].

Different researchers have addressed specific performance metrics within a rehabilitation plan such as maximizing reliability and resiliency, as well as minimizing leakage [2] and failure risk [3]. In addition, the strategy should aim to maximize 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 [4]. Among the different objectives considered, there are generally the minimization of operating cost and the maximization of the system reliability.

The rehabilitation technique must evaluate the performance during the entire lifetime of the system by considering the most important factors that affect its behaviour such as the ageing of the pipes or components of the WDN, the temporary unavailability of some components (e.g. pipe or pump), the increase of the losses as well as of the user water request. During these extended period analyses many cases of pressure deficit may occur, and to correctly reproduce the condition of insufficient nodal head the hydraulic simulation should be carried out by a pressure-driven solution. However, most of the rehabilitation approaches described in the literature are based on a classical demand-driven solution which is not suitable for the extended period simulations when, under critical operating conditions, the pressure may be smaller than the value of service. Alternatively, for solving the low head problem the use of “emitters” is combined with the classical demand-driven model (EPANET2) but this methodology may lead to incorrect solutions [5]. Finally, a pressure-driven modelling generally requests to define and introduce such a pressure-demand relationship.

This work presents a rehabilitation decision-making approach which allows to take into account both the hydraulic and mechanical deterioration processes of the elements constituting the system by solving the cases of insufficient head that may occur in extended period simulations. The hydraulic analysis is performed by a numerical algorithm that integrates the simulator by Todini and Pilati [6] and the iterative procedure by Todini [5] to solve the hydraulic problem. In this way, there is no need for an explicit relationship between pressure and demand, but the WDN problem when the head is insufficient may be solved by using a recursive approach that converges in few iterations. The pressure-driven hydraulic model permits to correctly model the pressure dependent outflows to users, but also to include the effect of leakage. In this way, we can analyse the influence of the leakage level on the system performance and evaluate the impact of different leakage percentages on the rehabilitation measures.

The rehabilitation decision model is probabilistic and it is based on a Monte Carlo (MC) technique. The lifetime of the WDN is represented as a series of normal and failure states of all the elements (sample life). In each sample life a forecast of future events related to the mechanical failure of pipes is carried out in probabilistic terms by the knowledge of the probability density function (pdf) of failure and repair times of each network component. The ageing of the system is taken into account by increasing the roughness of the pipes and also their failure rate. From the statistical analysis of the results for a large number of Monte Carlo simulations, the system reliability is provided in terms of water volume undelivered to users.

In addition, the evaluation of the reduction of the volume undelivered to users that may be obtained by replacing the pipes, can be useful in practical applications for decision makers to prioritize the pipes to be replaced. Finally, the optimal time to pipe replacement can be scheduled from the analysis of the variation of hydraulic benefits related to the variation of the replacing time of each pipe.

2 THE PROBABILISTIC APPROACH

2.1 Methodology

The proposed approach for the reliability assessment is based on the use of Monte Carlo simulations. The time of out-of-order T_f and the time T_r for the system to return to operation are random variables with probability density function (pdf) $f(T_f)$ and $g(T_r)$ respectively. From the knowledge of the pdf of failure and repair times of each network component, a sample life cycle of

the system may be reproduced as a succession of normal and failure states of all the elements. The entire succession constitutes a single Monte Carlo simulation (MCS).

Each MCS consists of three steps. 1) For each pipe the time to failure T_f is generated based upon the assumed distribution. The pipe with minimum time to failure is taken to be failed. 2) The hydraulic simulation is run and the computed hydraulic quantities necessary for the reliability evaluation are stored. 3) For the failed pipe a time to repair T_r is generated; whereas, for the other pipes, a new value of time to failure is generated. Such a new generation of times is needed when the failure rate increases with time, by considering the deterioration process of pipes. If the time to repair of failed pipe is longer than the smaller time to failure of the pipes, the number of the pipes unavailable increases of one unit. Otherwise, all the components will result normally operating. Steps 1-3 are repeated until the end of the lifetime. After all, the indicator chosen to represent the system performance is computed. When the number of realizations reaches the given maximum value, the output statistics are computed to provide the system performance estimate.

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 as sum of the water delivered to the users Q_{e_i} and the leakage allocated to the nodes Q_{l_i} . The latter is calculated as [7]:

$$Q_{l_i} = C_{l_i} H_i^b \quad (1)$$

where C_{l_i} , and H_i are respectively the leakage coefficient and the nodal pressure, with $0.5 < b < 1.5$.

Different pdfs of times to failure and repair may be assumed and the influence of the choice of the probability distributions on the reliability assessment was evaluated in [8]. By assuming an exponential distribution function for the time to failure pdf, if the failure rate λ is constant in time, the mean time to failure is $MTTF = 1/\lambda$.

The MC procedure allows for the analysis of the aspects related to the mechanical failure of pipes, also considering the ageing of pipes. Since the pipes deteriorate, the MTTF increases in time by assuming an exponential law $\lambda(t) = \lambda(t_0) \exp[A(t - t_0)]$, where $\lambda(t_0)$ is the failure rate at time $t = t_0$ and A is the increase coefficient. Moreover, the relationship $\varepsilon(t) = \varepsilon_0 - \exp(\beta t)(\varepsilon_0 - \varepsilon_n)$ was used for the pipe roughness at time t , where β is the coefficient that describes the rate of increase of the roughness from the initial value ε_n to the final one ε_0 . The initial value of the mean time to failure of each pipe may be given by the relationship existing between the mean time to failure and the pipe diameter, the latter being one of the pipe characteristics that has the largest impact on the breakage rate.

2.2 The pressure-driven solution

The hydraulic analysis is performed by a numerical algorithm that integrates the simulator by Todini and Pilati [6] and the iterative procedure by Todini [5] for solving correctly the cases of insufficient head that may occur in some operating conditions.

By this method there is no need to assume a relationship between the nodal flow and pressure $Q=f(H)$. The nodes with pressure deficit are working as reservoirs. If the head is below the minimum to grant a delivery, no water will be drawn out from the taps; if the head is insufficient for delivering the actual demand, a reduced demand may be satisfied. The procedure may be resumed in the following steps. 1) The WDN problem is solved in the demand-driven manner. If all the nodal heads are larger than the service level, the solution is assumed as correct. 2) If some nodes show an insufficient pressure, a new problem is solved setting the nodal pressure at the service level

in non-satisfied nodes, and the maximum demand Q_{e,max_i} compatible with this constraint is computed. 3) Some constraints must be replaced: the original constraint is set back in terms of demand Q_{e_i} at the nodes where $Q_{e,max_i} \geq Q_{e_i}$ since there is enough pressure at that node to satisfy the entire demand; the head constraint must be retained at the nodes where $0 \leq Q_{e,max_i} < Q_{e_i}$ since the pressure is insufficient to deliver the entire demand and the users will try to draw out as much water as possible; the constraint is set back in terms of demand equal to zero at the nodes where $Q_{e,max_i} < 0$, since the pressure is insufficient to provide any water.

The procedure converges to the right solution and generally there is no need to add more than three steps to the described procedure [5].

2.3 Reliability Assessment

The definition of a WDN failure is here based on inadequate delivery flow and a failure is deemed to occur during all the time period T_k when at node i the supplied flow $Q_{d,i,k}$ is lower than the nodal demand $Q_{r,i,k}$.

The nodal undelivered volume may be computed as:

$$Vu_{i,k} = (Q_{d,i,k} - Q_{r,i,k})T_k. \quad (2)$$

In the case of an extended period simulation, a single value can be calculated at each network operation instant. Therefore, the characterization of the system for the whole lifetime at each MCS can be carried out by calculating the total undelivered volume to users as:

$$Vu(mc) = \sum_{i=1}^N \sum_{k=1}^K (Q_{d,i,k} - Q_{r,i,k})T_k \quad (3)$$

where N is the number of supply nodes of the system and K is the number of time periods where the lifetime is subdivided.

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

$$EVu = \frac{1}{NMC} \sum_{mc=1}^{NMC} Vu(mc)$$

where NMC is the maximum value of MC runs. (4)

3 CASE STUDY

The analysis was applied to the synthetic water distribution network of Anytown modified as in [8]. In the examples the network scheme is simplified as shown in Figure 1. The node elevations, the mean nodal demands, the pipe data and the mean roughness are reported in [8]. The MTTR value is set constant for all the pipes, while the MTTF is a function of the pipe diameter.

The distributions of the time to repair are deduced from the analysis of recorded data in the water distribution network of Marghera (VE), in Italy [8]. The time to repair distribution is described by a log-normal probability distribution; labelled as $T = \ln t$ the logarithm of the time to repair, mean and variance are 2.93 and 0.362, respectively. On the basis of the available information about the pipe breaks occurred at the Marghera network, it was possible to deduce a relationship existing between the mean time to failure and the pipe diameter, being the latter one of the pipe characteristics that

has the largest impact on the breakage rate. For the mean time to failure the following expression is obtained:

$$MTTF_j = [0.2688 \exp(-0.0023D_j)]^{-1} \tag{5}$$

where D_j is the diameter of the j -th pipe, given in millimeters.

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 period of simulation was set equal to 30 years and 500 MCS were developed to ensure the convergence of the required statistics. The coefficient A is set to 0.1 years⁻¹ and the coefficient b is taken to be of 0.15 years⁻¹. The initial values e_n are reported in Table 3, while the final values e_o are obtained doubling the initial ones.

The effects on the system performance of three leakage percentages are analysed by considering a single leakage coefficient Cl_i for all nodes and $b=1.15$. Three different leakage percentages equal to approximately 15%, 30% and 55% of the design demand are analysed [9]. For sake of simplicity, in the developed examples the annual water demand as well as the leakage percentage are assumed to be constant along the network lifetime.

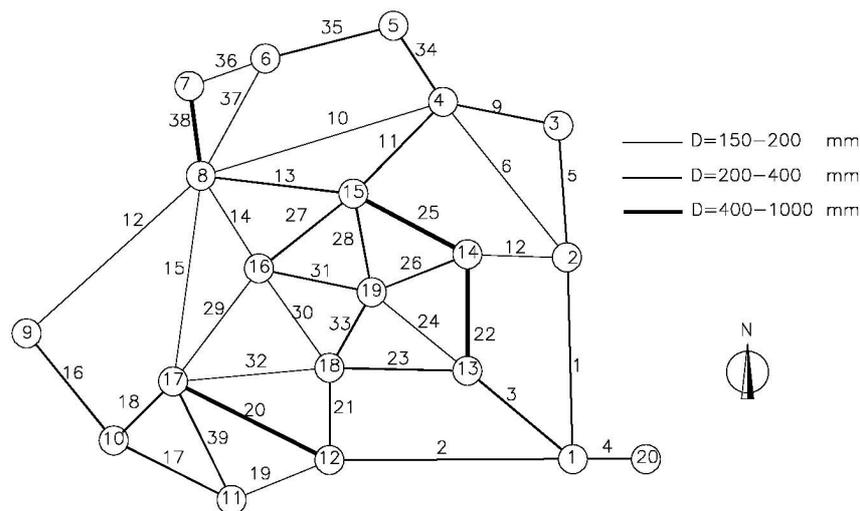


Figure 1. Scheme of the water distribution network Anytown (modified)

4 DISCUSSION ON THE RESULTS

By the statistical analysis on the 500 MC simulations the expected value of the undelivered volume to users is computed for each pipe given by (4) during the time of unavailability. The undelivered volume associated to the mechanical failure of every single pipe is reported in Figure 2 for the maximum leakage level and it is compared with the mean number of the breaks occurred for each pipe in the entire lifetime of the system. As expected, there is no correspondence between the number of breaks and the hydraulic deficit. In fact, the Anytown network is a redundant system and in case of unavailability of one or more pipes the water follows a different path to reach the supply nodes. According to equation (5) the conduct which presents the maximum number of breaks are pipes 24 and 36, as the failure rates are much higher for smaller diameters. These pipes present a mean value of breaks equal to 18 in 30 years, while the other pipes break less frequently. To pipe 4

a very large MTTF was assigned and it doesn't present any break. On the contrary, the pipe which in case of unavailability leads to the maximum undelivered volume to users is number 20, that is the pipe with the maximum diameter. Anyway, all the pipes in case of mechanical failure give a certain hydraulic deficit.

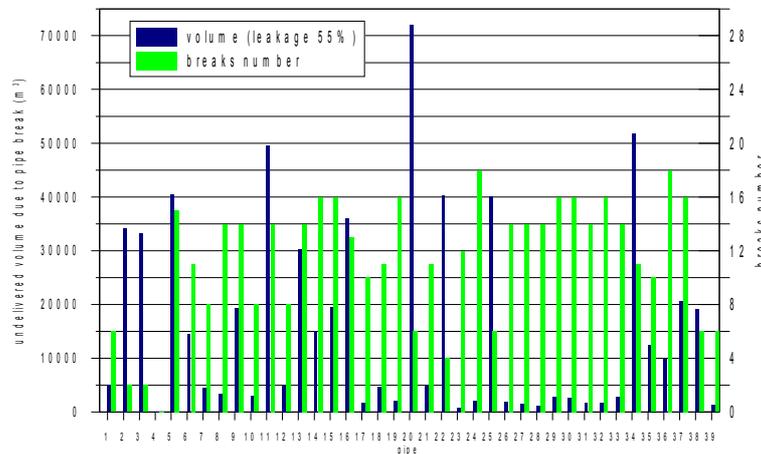


Figure 2. Expected value of undelivered volume (m³) associated with the break of each pipe and expected value of the pipe breaks number.

The influence of the leakage level on the hydraulic unavailability is shown in Figure 3 where the expected value of the undelivered volume associated with the mechanical failure of each pipe is reported for three leakage percentages. If the leakage level is 15%, only few pipes give hydraulic failure. The pipe which contributes for a 20% to the total undelivered volume of the system is number 20. Also pipes 34 and 16 concur to the hydraulic deficit in significant manner. The pipe with the smallest contribution is number 11. When the leakage level increases the number of pipes concurring to the undelivered volume to users increases and for a percentage of 55% all the pipes, with exception of number 4, have a role in the total hydraulic failure. This means that the contribution of pipes 20, 34 or 16 to the total deficit reduces and tends to become similar to other pipes. For instance, the contribution of pipe 16 is smaller than that of pipe 11. This aspect explains the results of Figure 4 where the reduction of the total undelivered volume of the system obtained with the replacement of some pipes, one at a time, is illustrated for different years of intervention.

For a smaller level of leakage the maximum benefit in terms of reduction of the undelivered volume is obtained by substituting pipe 20 with a maximum value of 14%. A minor advantage is given by the substitution of pipes 34 and 11, while no benefits are gained with the substitution of pipe 16. When the leakage level is 55% the replacement of pipe 20 leads to a maximum reduction of the total undelivered volume of the system about 8%, value smaller than that computed in the case of smaller percentage. The impact of the substitution of pipe 20 is similar to that of pipe 34 and larger than that of pipe 16. The replacement of pipe 11 seems to give the largest benefit, having a maximum reduction of about 10%.

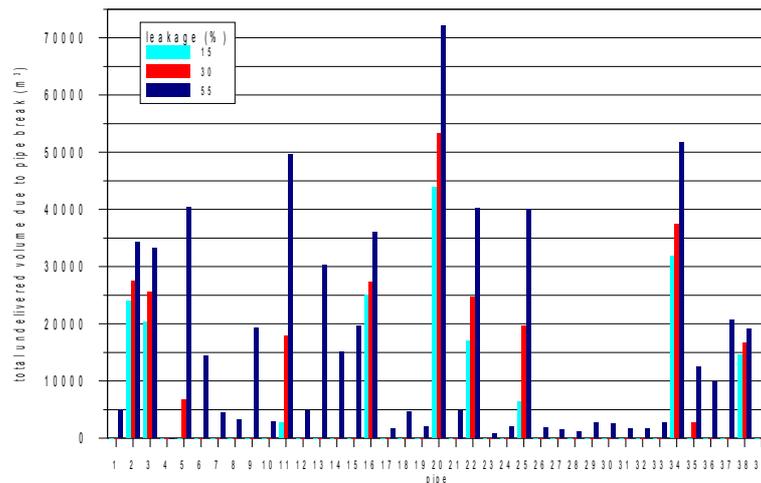


Figure 3. Expected value of undelivered volume (m^3) associated with the break of each pipe for different leakage levels.

From the behaviour of the total undelivered volume reduction with the replacement time it is possible to schedule the optimal year of intervention for improving the system performance. In all the cases here reported the optimal year to replace pipes is between 15 and 20 years. Obviously, an economic analysis should be applied to integrate these results with cost-effective criteria. Moreover we want to stress that the results here obtained are related to a spatially uniform distribution of the leakage coefficients Cl_i , and that any localized leakage condition may lead different results.

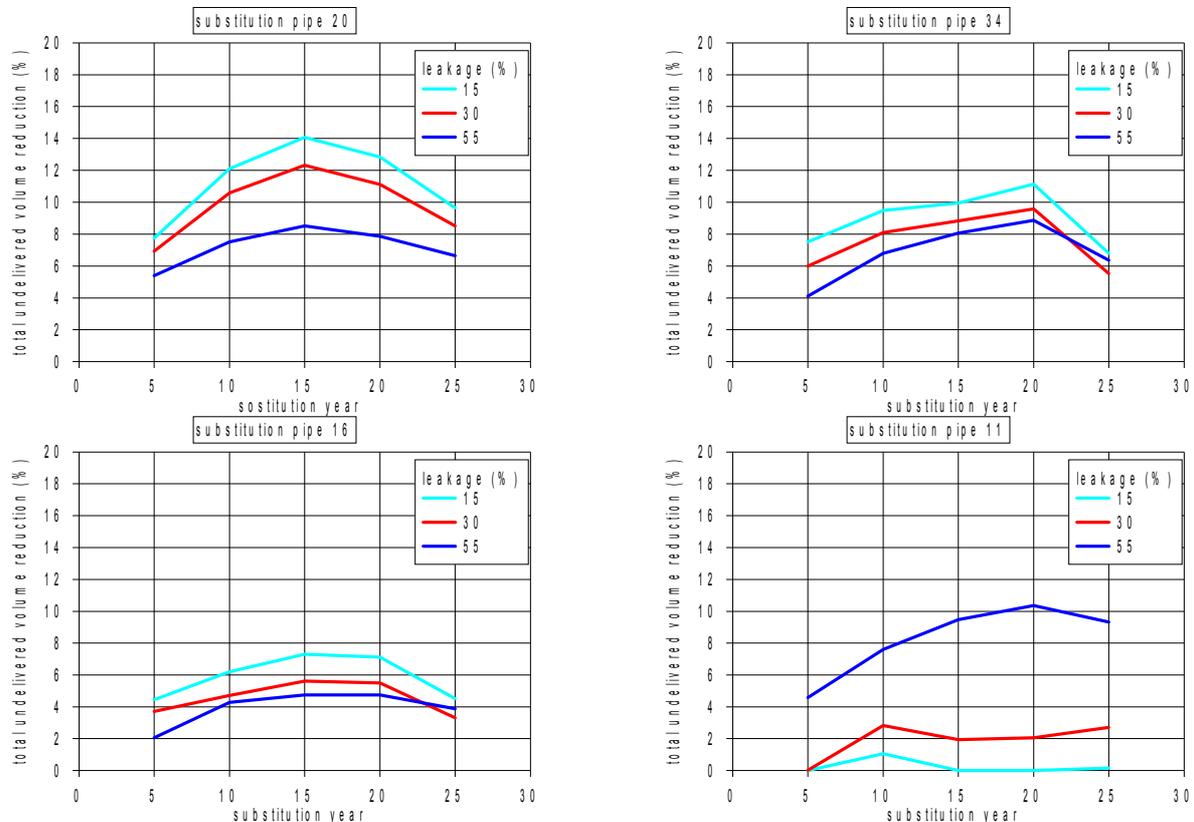


Figure 4. Reduction (%) of the expected value of the total undelivered volume by replacing pipe 20 (a), 34 (b), 16 (c) and 11 (d) as a function of the substitution year.

5 CONCLUSIONS

A model for supporting the WDN rehabilitation planning was proposed to assess the intervention priorities and the optimal times for the pipe replacement as the percentage of water leakage, that in the example is assumed affects all the network pipes, varies in the system.

The leakage percentage can affect the planning of WDN rehabilitation. For smaller leakage level, 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 not provided to users because of their mechanical unavailability even if they break rarely. In the case of larger leakage percentages, the same benefits can be obtained also by replacing pipes characterized by a smaller diameter and a larger number of breaks. This choice leads to minor replacement costs and permits to reduce the frequent disruptions of supply to users.

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