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*Original*

Performance Assessment of Water Distribution Systems Subject to Leakage and Temporal Variability of Water Demand / Darvini, Giovanna; Ruzza, V.; Salandin, P.. - In: JOURNAL OF WATER RESOURCES PLANNING AND MANAGEMENT. - ISSN 0733-9496. - 146:1(2020). [10.1061/(ASCE)WR.1943-5452.0001143]

*Availability:*

This version is available at: 11566/271766 since: 2024-04-29T17:04:23Z

*Publisher:*

*Published*

DOI:10.1061/(ASCE)WR.1943-5452.0001143

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1 **Performance assessment of water distribution systems subject to leakage and temporal**  
2 **variability of water demand**

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8 **Abstract**

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

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

## 27 **Introduction**

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

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

48 WDN reliability is generally assessed through performance indicators that relate the water  
49 delivered to users to their demands under critical operational scenarios due to either mechanical

50 or hydraulic failure. Therefore, in the evaluation of the system performance during a prescribed  
51 time horizon, all the factors affecting its continued reliability should be considered, such as the  
52 ageing of pipes or components, and the temporary unavailability of some components, e.g.  
53 pipes or pumps (e.g. Kleiner et al., 1998; Mazumder et al., 2019). During these extended period  
54 analyses the pressure head may often prove to be insufficient and that in many cases of pressure  
55 deficit may occur. To correctly reproduce the condition of insufficient nodal head the hydraulic  
56 simulations should be carried out by a pressure-driven solution (e.g. Giustolisi et al., 2008).

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

73 Among factors connected with the water demand behaviour, temperature is the most  
74 relevant because it directly influences several sources of water consumption such as showers

75 or water for gardens. Water consumers also respond to the occurrence of rainfall and other  
76 climate variables, though rainfall seems to have a dynamic effect, in the sense that it reduces  
77 water demand initially, but the effect diminishes over time (e.g. Herrera et al. 2010).

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

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

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

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

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

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

## 112 **Assessment of reliability in WDSs subject to leakage and temporal variability of the** 113 **user demand**

### 114 *Probabilistic approach in the Extended Period Simulations (EPS)*

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

121 The MC procedure allows for the analysis of the aspects related to the mechanical failure of  
122 pipes, also considering their ageing. Different pdfs of failure and repair times could be  
123 assumed, and the influence of the choice of the probability distributions on the reliability  
124 assessment was evaluated in Darvini (2014). Under the assumption of an exponential pdf the

125 mean time to failure is  $MTTF = 1/\lambda$ , where the failure rate  $\lambda$  is commonly assumed constant.  
126 Since the pipes deteriorate, the failure rate increases in time according to the exponential law  
127 given by Shamir and Howard (1979)

$$128 \quad \lambda(D,t) = \lambda(D,t_0)\exp[A(t - t_0)] \quad (1)$$

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

134 Defined as  $\varepsilon_n$  (m) the initial roughness of the pipe when it was new ( $t=t_0$ ) and  $\varepsilon_o$  (m) the  
135 constant roughness characterizing the aged pipe, the pipe roughness behaviour during time is

$$136 \quad \varepsilon(t) = \varepsilon_o - \exp(\beta t)(\varepsilon_o - \varepsilon_n) \quad (2)$$

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

141 Each MCS consists of three steps. 1) At the beginning of the simulation, all the system pipes  
142 are normally operating. For each pipe the time to failure  $T_f$  is generated based upon the assumed  
143 exponential probability distribution at the time  $t$ . Among all the pipe of the system, the one  
144 with the shortest time to failure is considered out of work, and the current time is increased of  
145  $T_f$ . 2) The hydraulic simulation is run and all the computed hydraulic quantities required for  
146 the reliability evaluation are stored. 3) Due to the deterioration process of pipes, the failure rate  
147  $\lambda$  increases with time, and a new generation is needed. A time to repair  $T_r$  is generated for the  
148 out of work pipe, whereas for the other pipes a new value of time to failure  $T_f$  is generated. If  
149 the  $T_r$  of the failed pipe is longer than the smaller  $T_f$  of the other operating pipes, the number

150 of the unavailable pipes increases by one unit. Otherwise, all the components will result  
151 normally operating. Accordingly, the current time is increased of the minimum among the  $T_r$   
152 and all the  $T_f$ . Steps 1-3 are repeated until the end of the planned horizon, then the quantities  
153 needed to compute performance index are evaluated and stored. When the number of  
154 realizations reaches the given maximum value, the statistics on the stored values are calculated  
155 to provide the system performance estimate.

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

#### 160 *Leakage model*

161 In the pressure-driven solution the outflows  $Q_i$  at the unknown head nodes are assessed as a  
162 function of the nodal demand and pressure head, and leakage can be accurately modelled. The  
163 nodal flow rate  $Q_i$  can be calculated as the sum of the water delivered to the users  $Q_{s_i}$  and the  
164 leakage allocated to the nodes  $Q_{l_i}$ .

165 The leakage model focuses on representing the background losses resulting from leaks that  
166 are neither detectable nor locatable, and the undetectable leaks which are however locatable  
167 through leakage detection campaigns. These types of leaks are of low intensity but persist over  
168 time without being identified, thus resulting in high volumes of water losses. On the contrary,  
169 detectable leaks are associated with breaks in medium or large-sized pipes and they are  
170 generally located and eliminated in a short time by repairing the damaged pipe. Therefore,  
171 though they may be characterized by high flow losses, their brief duration means that relatively  
172 small volumes will be lost overall (Alvisi and Franchini, 2009). Despite this fact, due to the  
173 failure of one or more pipes, a pressure shortfall may affect a portion of the WDN, and in that

174 area the nodal demand may not be met along all the time needed to repair the damaged  
175 elements.

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

$$178 \quad Q_{l_i} = C_{l_i} H_i^{N_{l_i}} \quad (3)$$

179 where  $Q_{l_i}$  is the leakage flow rate at  $i$ -th node,  $H_i$  is the pressure head at  $i$ -th node,  $C_{l_i}$  is the  
180 leakage model coefficient and  $N_{l_i}$  is the leakage model exponent.

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

#### 184 ***Reliability Assessment***

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

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

$$190 \quad Vu_{i,k}(T_k) = [Q_{d_{i,k}}(T_k) - Q_{s_{i,k}}(T_k)] T_k \quad (4)$$

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

$$194 \quad Vu(mc) = \sum_{i=1}^N \sum_{k=1}^K [Q_{d_{i,k}}(T_k) - Q_{s_{i,k}}(T_k)] T_k \quad (5)$$

195 where  $N$  is the number of supply nodes of the system and  $K$  is the number of time periods  
196 in which the analysed lifespan is subdivided.

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

199 
$$EV_u = \frac{1}{NMC} \sum_{mc=1}^{NMC} V_u(mc) \quad (6)$$

200 where NMC is the maximum value of MC runs.

201 Through the Monte Carlo analysis it is possible also to evaluate  $EV_{u_j}$ , that is the contribute  
 202 given by the j-th pipe to the total undelivered volume  $EV_u$ .

203 **Case study**

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

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

216 
$$MTTF(D_j) = [0.2688 \exp(-0.0023 D_j)]^{-1} \quad (7)$$

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

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

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

232 The effects on the system performance of three leakage levels (13%, 25%, 50%) are  
233 analysed by considering a single leakage coefficient  $Cl_i$  for all the nodes and  $N1_i=1.0$  (eq. 3),  
234 being the leakage percentages chosen according to recent reports on the leakage behaviour in  
235 Italy (ISTAT 2018). In the following examples the mean annual water demand as well as the  
236  $Cl_i$  and  $N1_i$  coefficients are assumed constant during the planning horizon. The temporal  
237 variation of the user demand is represented by the three hourly patterns p1, p2, p3 illustrated  
238 in Figure 2 (Milano 2012). The pattern p1 is characterized by a large discrepancy between the  
239 water demand at each hour during the day and the constant daily demand, thus showing higher  
240 hourly coefficients. These hourly coefficients vary between 0.2 and 1.56 within 24 hours, the  
241 minimum occurs between 2 and 3 a.m. and the maximum at 12 p.m. The pattern p2 presents  
242 reduced fluctuations respect to p1, varying the hourly coefficients between 0.4 and 1.38. The  
243 pattern p3 is closer to the constant pattern p0, with limited fluctuations bounded in the range  
244 0.52-1.32.

## 245 **Discussion on the results**

### 246 ***Influence of the leakage level on the hydraulic deficit***

247 The expected value  $EV_u$  of the volume undelivered to users during the planning horizon  
248 volume is illustrated in Figure 3 as a function of the level of leakage for the pattern p0, p1, p2,

249 and p3 considered. When pipes are broken the nodal pressure could be insufficient to satisfy  
250 the request of the users and a fraction only of the water demand can actually be delivered. We  
251 stress that EVu is a performance index affected by leakage, but it does not account for the  
252 volume of leakage. In other words, an identification of the leakage positions (e.g. Ruzza et al.,  
253 2015) is outside of the aim of the present work, and the pipes leading to the greatest EVu values  
254 are not necessarily affected by maximum leakage.

255 From a general point of view, the influence of the pattern is more significant for smaller  
256 leakage levels than for a large percentage of leakage. For a leakage level of 5% the expected  
257 value of the undelivered volume increases of about 600% from the pattern p0 to p1, while for  
258 a leakage level of 50% EVu increases of about 100%. The hydraulic deficit is at the minimum  
259 in the absence of leakages and increases with the leakage level in non-linear manner. For a  
260 leakage level from 5% to 50% the undelivered volume increases from  $1 \times 10^4 \text{ m}^3$  to  $8 \times 10^4 \text{ m}^3$   
261 for a constant pattern, while for p1 the undelivered volume increases from  $7 \times 10^4 \text{ m}^3$  to  $16 \times 10^4$   
262  $\text{m}^3$ .

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

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

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

281 In the planning horizon considered, pipes 2, 3 and 22 show less than 5 breaks, eight pipes  
282 (1, 7, 10,12, 20, 25, 38 and 39) have more than 5 but less than 10 breaks, while all the remaining  
283 pipes show more than 10 but less than 20 breaks.

284 Thanks to the system redundancy, the mechanical unavailability does not lead necessarily  
285 to hydraulic failure, the latter one occurring when the nodal pressure is lower than the service  
286 level and thus the nodal demand cannot be fully accomplished. The expected value of the  
287 undelivered volume ( $m^3$ ) due to the breaks of each pipe in the planning horizon of the system  
288  $EV_{u_j}$  is illustrated in Figure 5 for different leakage levels in the case of both constant pattern  
289  $p_0$  and time-variable pattern  $p_1$ . The comparison between the undelivered volume associated  
290 to the mechanical failure of every single pipe and the mean number of the breaks occurred on  
291 that pipe in the planning horizon shows that the hydraulic deficit  $EV_{u_j}$  is not proportional to  
292 the number of  $j$ -th pipe breaks. In fact, the Anytown network is a redundant system and in case  
293 of unavailability of one or more pipes the water follows different paths to reach anyway the  
294 supply nodes.

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

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

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

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

315 To assess the intervention priorities among all the pipes subject to failure and help the  
316 utilities in the management of the distribution systems, the contribution of each pipe to the  
317 inability of the system to satisfy the user request in terms of delivered water is analysed. Figure  
318 6 shows the percentage ratio between the expected value  $EV_{u_j}$  of the undelivered volume  
319 consequent to the breaks of every single pipe and the expected value of the total undelivered  
320 volume of the system  $EV_u$ .

321 For a constant demand pattern and no leakage three pipes concur to the hydraulic deficit of  
322 the system and pipe 38 is responsible for more than 50% of the total undelivered volume. When

323 the leakage level is 13%, the pipes which contribute for more than 60% to the total undelivered  
324 volume of the system are number 2 and 38. Also pipes 3 and 16 significantly concur to the  
325 hydraulic deficit. The pipe giving the smallest contribution is number 34. As the leakage level  
326 increases the number of the pipes concurring to EVu increases, and the impact of each single  
327 pipe on the total hydraulic deficit reduces. The contribution of pipes 2, 3 and 38 to the total  
328 deficit reduces and tends to become similar to others. For instance, for a leakage level of 50%  
329 the contribution of pipe 3 is smaller than that of pipe 34.

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

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

#### 340 ***Influence of the demand pattern on the hydraulic deficit***

341 The influence of the hourly pattern of the water demand is illustrated in Figure 7, where the  
342 EVu<sub>j</sub> is shown for different demand patterns and in the leakage level of 50%, besides the no  
343 leakage case.

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

346 When the water demand is constant during the day (pattern p0), some pipes don't produce  
347 hydraulic deficit as a consequence of their mechanical unavailability, but when an hourly

348 variability is introduced in the demand pattern this is no longer true. Some pipes, like pipe 25  
349 in the no leakage case, lead to hydraulic deficit in the system only for the extremely variable  
350 pattern p1.

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

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

### 363 ***Reduction of the hydraulic deficit by pipe replacement***

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

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

373 daily demand pattern p0.

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

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

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

395 The reduction of the undelivered volume to users obtained by replacing the same pipes in  
396 the case of the pattern p1 are reported in Figure 9. As expected, the substitution of the pipe  
397 gives a smaller benefit in terms of reduction of the undelivered volume to users in comparison

398 with the case of constant demand pattern. Thus, the replacement of a specific pipe does not  
399 produce a benefit larger than that of a different substitution. This is evident for the case of a  
400 leakage level of 13% where the replacement of the pipes 2 and 38 produces a benefit between  
401 8% and 12%, significantly smaller than the corresponding benefit obtained for the pattern p0.  
402 For the pipe 16 also the benefit obtained by the substitution reduces as the pattern becomes  
403 variable in time. On the contrary, for the pipe 20 the advantage gained with the substitution  
404 significantly increases when the pattern changes from p0 to p1. Thus, for a leakage percentage  
405 of 13%, the larger hydraulic benefit is reached with the replacement of the pipe 20, while in the  
406 case of the pattern p0 the break of the same pipe doesn't produce any hydraulic deficit and the  
407 only advantage of its substitution is related to the reduction of the pipe roughness. When the  
408 leakage level increases up to 50% the advantage of the pipe replacement decreases respect to  
409 the case of smaller leakage percentage and the differences between two cases are not relevant.

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

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

## 421 **Conclusions**

422 The influence of the temporal variability of water demand on the evaluation of the performance

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

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

439 The contribution of each pipe on the EVu is also considered. For a constant pattern only few  
440 pipes, but among the largest in terms of diameters, concur to the whole hydraulic deficit, though  
441 they are affected by a limited number of breaks. Thus, the substitution of these pipes provides  
442 a large benefit in terms of reduction of EVu, and this occurs mostly when the leakage  
443 percentage is limited. As the leakage level increases, the number of pipes that - due to the  
444 mechanical failure - yield hydraulic deficit increases, and equivalent advantages in term of  
445 reduction of EVu can be obtained by replacing also pipes characterized by small diameters and  
446 a larger number of breaks. This is more manifest with patterns highly variable in time when, in  
447 case of breakage during the periods of maximum demand, not only largest pipes are responsible

448 for situations of hydraulic deficit, and no conduit plays a fundamental role with respect to the  
449 others. Thus, the priority in the replacement can be assigned also to pipes characterized by  
450 smaller diameter and large number of breaks, with the advantage to reduce the inconvenience  
451 for users related to the number of repeated repairs.

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

#### 462 **Data availability**

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

#### 465 **Notation**

466 *The following symbols are used in this paper:*

467  $A$  = coefficient of breakage rate growth

468  $Cl_i$  = leakage model coefficient at i-th node

469  $D_j$  = diameter of the j-th pipe

470  $EV_u$  = expected value of total undelivered volume

471  $EV_{u_j}$  = expected value of total undelivered volume due to the breaks of j-th pipe

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

473  $g(T_r)$  = probability density function of the time to repair  
 474  $H_i$  = pressure head at i-th node  
 475  $H_{s_i}$  = service pressure heads at i-th node  
 476  $K$  = total number of states  
 477 MCS = simulation Monte Carlo  
 478 MTTF = mean time to failure  
 479 MTTR = mean time to repair  
 480  $N$  = total number of nodes  
 481  $N_{l_i}$  = leakage model exponent at i-th node  
 482 NMC = maximum number of Monte Carlo simulations  
 483  $Q_i$  = flow rate at i-th node  
 484  $Q_{d_{i,k}}$  = water demand at i-th node in the k-th state  
 485  $Q_{l_i}$  = leakage flow rate at i-th node  
 486  $Q_{s_i}$  = flow rate delivered to users at i-th node  
 487  $Q_{s_{i,k}}$  = flow rate delivered to users at i-th node in the k-th state  
 488  $t$  = current time  
 489  $t_0$  = initial time  
 490  $T_f$  = time to failure  
 491  $T_k$  = duration of k-th state of hydraulic failure  
 492  $T_r$  = time to repair  
 493  $V_u(mc)$  = total undelivered volume in the mc-th Monte Carlo simulation  
 494  $V_{u_{i,k}}$  = undelivered volume at i-th node in the k-th state  
 495  $\beta$  = roughness growth rate  
 496  $\varepsilon$  = pipe roughness  
 497  $\varepsilon_n$  = initial roughness of the pipe

498  $\epsilon_0$  = roughness of the aged pipe

499  $\lambda$  = 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							