Simplified Procedure for Water Distribution Networks Reliability Assessment

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Abstract: This work describes a simple procedure for the analysis of water distribution system (WDS) performance. It is based on the reliability assessment of a WDS during failure states resulting from the unavailability of a pipe (maintenance or repair), taking into account the probability of the failure events. The procedure may consider changes in daily demand by using patterns defined to represent seasonal trends. Applying this method to a network with around 200 pipes demonstrates its usefulness, especially when the aim is to objectively compare different design solutions. The procedure takes also into account variations in design parameters such as an increase in water demand and/or a drop in hydraulic conductance resulting from pipe lining degradations (corrosion or deposits). **DOI: 10.1061/(ASCE)WR.1943-5452.0000184.** *© 2012 American Society of Civil Engineers.*

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Introduction

The water services industrialization process, which has been under development for several years in a number of countries, requires modern analysis and design tools to define maintenance and management strategies on the basis of optimizing the cost/benefit ratio.

In this context, WDS performance analysis is frequently required, both to diagnose existing systems and to design new networks or improve old ones.

The primary performance of a WDS is water distribution, with sufficient nodal pressure to satisfy demand under all operative conditions, which could reasonably occur during its lifetime.

According to this definition, the aim of performance analysis is not only to evaluate if the required performance will be met, but also to quantify to what extent the performance is satisfactory.

With regards to only hydraulic aspects (water quality problems are usually studied independently), it is possible to use two different approaches.

The first, the deterministic approach, simply verifies performance satisfaction referring to a limited number of working conditions (design conditions). If, for all prefixed design conditions, the system is able to guarantee the performance required the analysis result is positive. When the response is unsuitable, analysis indicates elements that are useful to define technical improvements.

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This approach, even though commonly used, has some limits. The number of scenarios considered in the analysis is very limited, and it is impossible to assign them a probability value. Furthermore, the analysis shows whether the system is suitable or not, but it is not able to give any indications of to what extent and, therefore, it is not possible to compare different design solutions or plants.

To quantify WDS performance, it is necessary to adopt a probabilistic approach that can evaluate system behavior with reference to the many different working conditions not usually considered in the design phase but have a certain probability of occurring. Calculation involves statistic evaluation of performance indexes (generally defined according to the supply deficit), which are calculated by running hydraulic simulations of many different operative scenarios, each having been assigned its own probability value. The scenarios are defined by introducing, as suggested by Bertola et al. (2004), the system failure factors and their associated probability. The factors are as follows:

1. Mechanical factors such as interruption of service for maintenance or repair or breakage of pipes, pumps, and valves, and
2. Hydraulic factors such as random variations (both in time and in space) of water demand and/or a drop in pipe hydraulic conductance resulting from obsolescence.

The statistical processing of simulation results give an evaluation of performance satisfaction and effectiveness.

Probabilistic methodologies suggested in the literature are numerous and, in most cases, include failure caused by both hydraulic and mechanical factors (Khomshi et al. 1996; Tanymboh et al. 2001). Some use a simple PDA analysis, without taking into consideration a probabilistic approach to assess the reliability of the system, such as the ones proposed by commercial software. Moreover, the proposed calculation techniques differ greatly but are based on long-period simulations during the entire network lifetime, sometimes even for periods as long as 50 years (Bertola and Nicolini 2004), and calculations are generally made on an hourly basis. The uncertain elements are simulated in a stochastic way by using Monte Carlo techniques. The number of simulations necessary to guarantee convergence are hardly ever specified and the practical examples given in the literature refer to networks with

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a very small number of pipes and nodes. In addition, calculation takes a long time, in some cases even several days.

In many works, by way of suitable simplifications, authors try to solve the problem by running as few network hydraulic simulations as possible; some also try to reduce part of the problem to closed forms to then match the numerical simulations (Xu and Goulter 1998).

Currently, performance analysis of WDS by using a probabilistic approach is hardly ever carried out on systems consisting of a high number of pipes and nodes. Significant progress in the field of scientific research has been made but has not yet been put into practice. The consequence is that, despite the existence of an important conceptual improvement, the design procedure is still based on conventional and deterministic hydraulic analysis.

Some commercial software packages have attempted to move beyond the traditional Demand Driven Analysis approach, implementing simple and consolidated procedures for Pressure Driven Analysis. Although these packages calculate the actual delivered flow with a certain available pressure, they do not take into account the probability of a pipe outage and, therefore, do not consider the effects on network reliability.

In this work, the authors propose a simple procedure for WDS performance analysis, which can also be applied to systems with a large number of nodes and pipes, because of the relatively low number of hydraulic simulations required.

The methodology is based on the reliability evaluation of a WDS working under failure conditions resulting from the unavailability of components or to other factors that may affect the efficiency of the system, such as an increasing demand. It can be used to assess reliability in the design phase of a network and is especially useful when faced with comparing different solutions.

Description of Procedure

This methodology defines the reliability of an hydraulic system as its ability to satisfy users taking into account the various working conditions to which it may be subjected during its operative life.

The working conditions are defined by taking into account the temporary unavailability of some components resulting from mechanical failures (e.g., damaged electromechanical devices, pipes out of order for maintenance). In particular, according to many authors, only pipe failures are considered, and each failure event represents the unavailability of only one pipe; in fact, it has been shown (Su et al. 1987) that the concurrent unavailability of two or more components is a very low probability event. For this reason, reliability values vary very little considering the unavailability of one element at a time or the combined unavailability of more elements.

This is on the basis of the assumption that each pipe can be isolated (for maintenance or repair) by using valves located at its extremities. In practice, this is not always the case because, for economic reasons, the valves are not necessarily located at the end of each pipe is such that isolating one pipe often means isolating others.

Regarding this issue, Walski (1993) suggested using an approach involving "segments" of a distribution system (instead of pipes), which can be isolated with valves and are considered the basic unit for assessing reliability. Subsequently, other authors (Jun and Loganathan 2007; Kao and Li 2007; Giustolisi et al. 2008) provided methods to identify the segments that will be isolated once certain valves have been closed. Recently, Giustolisi and Savic (2010) and Creaco et al. (2010) have proposed methods for the optimal placement of isolation valves.

Even though the method proposed in this study considers the single pipe (not the segment) as the unit for assessing reliability, it can easily be adapted to consider "segments" as units once they have been identified.

According to this definition, and as suggested by several authors (among others, Gupta and Bhave 1994; Gargano and Pianese 2000; Tanymboh et al. 2001), reliability is calculated by using performance indicators on the basis of the ratio between volumes actually delivered W_A during the evaluation period and the volume W_R required by the users as follows:

$$R = \frac{W_A}{W_R} \tag{1}$$

This definition, which can be applied on a global scale or a local scale (at the single delivery node), has the advantage that the other indicators do not have; it has a simple physical significance which is immediately referable to the performance level experienced by the users.

Furthermore, in an industrial context, these types of indexes allow managers to evaluate how much more water could be sold if the network were to work better.

It is fairly common practice to define indexes by using flowrates instead of volumes. The indexes expressed in terms of flow maintain the significance (based on volume deficits) if the temporal intervals assumed in the calculation of requested and delivered volume are the same and constant.

Performance Indicators

For an assigned distribution network, with NN demand nodes (each node is identified by index j, with $j = 1$, NN), subject to different possible working states as far as the availability of its components is concerned (each working state is defined by index k with $k = 1$, NS where NS is the number of working conditions analyzed), consider a demand that varies over time defined by means of n flows ($Qr_{j,i}$) required at the nodes, each defined for the ith interval of time ($i = 1$, n) of duration Δt (with Δt constant for the n considered intervals).

In the procedure presented in this work, the demand is defined by only one demand pattern that describes the demand course in a typical day, assuming $\Delta t = 1$ h ($n = 24$) or $\Delta t = 2$ h ($n = 12$).

$Q_{j,i}$ is the flow actually delivered at the jth node, during time interval i, evaluated as a function of the head at the node ($Q_{j,i} = Qr_{j,i}$ if the nodal head is greater or equal to that necessary to meet the full demand; $0 < Q_{j,i} < Qr_{j,i}$ if the nodal head is less than that necessary to meet full demand).

Assume, finally, every considered working state to be constant during all analysis time ($n\,\Delta t$), which corresponds to a whole day. The effect induced by the unavailability of a component is calculated referring to water demand during the typical day; introducing the probability of a working state, it is possible to evaluate its effect during the plant's operative life.

With these assumptions, the following indexes can be formulated (Gargano and Pianese 2000; Ciaponi 2009), which represent, for a single demand node (or the entire network) and for each of the NS mechanical working conditions of the network, the ratio between the water volume actually delivered and the one required in the considered time range ($n\,\Delta t$) as follows:

$$R_{j,k} = \frac{\sum_{i=1}^{n} Q_{j,i,k}}{\sum_{i=1}^{n} Qr_{j,i}} \tag{2}$$

for each node : local indicators (number equal to NS)

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$$\mathrm{RR}_k = \frac{\sum_{j=1}^{\mathrm{NN}} \sum_{i=1}^{n} Q_{j,i,k}}{\sum_{j=1}^{\mathrm{NN}} \sum_{i=1}^{n} Qr_{j,i}} \tag{3}$$

for whole network : global indicator (number equal to NS)

Attributing to each working condition a weight w_k equal to its associated probability, it is, therefore, possible to calculate the following indexes:

$$R_j = \sum_{k=1}^{\mathrm{NS}} (R_{j,k}\, w_k) \qquad \text{local indicators for all working conditions} \tag{4}$$

$$\mathrm{RR} = \sum_{k=1}^{\mathrm{NS}} (\mathrm{RR}_k\, w_k) \tag{5}$$

global indicator for all working conditions

Indexes, Eqs. (4) and (5) represent, at a single node or whole network scale, the ratio between the water volume actually delivered and the one required in the considered time range ($n\,\Delta t$), with reference to different working states (in terms of component availability) that may occur during plant activity.

It is easy to understand how the indexes previously presented cannot fully illustrate the reliability of a system because water deficits of different severity in relation to the spatial and temporal distribution of failures, can determine the same index values. For this reason, supplementary indices have been introduced (Gupta and Bhave 1994), as described in the following.

1. Temporal factor, the ratio between the duration of acceptable situations and the total duration as follows:

$$F_t = \frac{\sum_{k=1}^{\mathrm{NS}} \sum_{j=1}^{\mathrm{NN}} \sum_{i=1}^{n} \beta_{j,i}\, \Delta t}{\mathrm{NS}\ \mathrm{NN}\ \ T} \tag{6}$$

- $\beta = 1$ if $Q_{j,i}/Qr_{j,i} \geq$ acceptable value (for example 0.5),
- $\beta = 0$ if $Q_{j,i}/Qr_{j,i} <$ acceptable value, and
- T = total duration of period.

2. Nodal factor, corresponding to the geometric mean of nodal indices

$$F_n = \left[\prod_{j=1}^{\mathrm{NN}} R_j\right]^{1/\mathrm{NN}} \tag{7}$$

Therefore, the global performance index RR, calculated by Eq. (5) is corrected as follows:

$$\mathrm{RR}_c = \mathrm{RR} F_t F_n \tag{8}$$

Applying corrective coefficients, the performance index Eq. (8) may still be read as a representative indicator of the average water deficit that will occur during the system's life; deficit values, however, are weighted according to their severity.

Probabilistic Evaluation of Mechanical Failures

To calculate the aforementioned performance indicators it is necessary to establish the probability (w_k) of a kth working state.

Because pipes can be repaired, probabilistic analysis of mechanical failure is based on the concept of availability, which includes both breakdown probability and the time required for repair (Khomsi et al. 1996; Gargano and Pianese 2000; Tanymboh et al. 2001). The availability A_l of each pipe l ($l = 1$, NT) is the probability that it will be available when necessary and can be evaluated by using the following equation:

$$A_l = \frac{\mathrm{MTTF}_l}{\mathrm{MTTF}_l + \mathrm{MTTR}_l} \tag{9}$$

where MTTF = mean failure duration; and MTTR = mean time to repair; these parameters are easy to obtain if the failure rate λ_l (number of annual breaks for each unit of pipe length) and repair rate μ_l are known as follows (inverse of mean time to repair):

$$\mathrm{MTTF}_l = \frac{365}{\lambda_l L_l} \tag{10}$$

$$\mathrm{MTTR}_l = \frac{1}{\mu_l} \tag{11}$$

where L_l = pipe length.

The complement to 1 of availability A_l is unavailability (U_l), defined as the probability that the pipe is unavailable

$$U_l = 1 - A_l = \frac{\mathrm{MTTR}_l}{\mathrm{MTTR}_l + \mathrm{MTTF}_l} \tag{12}$$

Once A_l and U_l have been established, probability values associated to the different working states of the network can easily be calculated.

The probability that the network is fully functional (no pipes out of order) is given by the following expression:

$$p(0) = \prod_{l=1}^{\mathrm{NT}} A_l \tag{13}$$

The probability that pipe l (and only pipe l) is unavailable is

$$p(l) = p(0)\,\frac{U_l}{A_l} \tag{14}$$

The probability values given by Eqs. (13) and (14) can be used in Eqs. (4) and (5) as weight coefficients w_k to compute the performance indicators obtained for the different system working conditions.

To define MTTF and MTTR, failure rate λ and repair rate μ are required.

Usually, the failure rate λ is assumed constant in time as described by the well-known bath- curve, which illustrates how, after a brief initial period when the component can experience construction defects that did not appear during inspection, the failure rate drops to values that remain more or less constant for most of its life (during which breakdowns are essentially random). The rate then increases when the system is old because of breakages caused primarily by wear and tear.

The failure rate depends on several factors, which are often related to local and/or particular situations. Failure rate values published in the literature range from 0.05 failures/(km year) [0.08 failures/(mile year)] to 1 failure/(km year) [1.6 failures/(mile year)] with a large dispersion (particularly big for small diameters), which in general shows a decreasing trend of λ with diameter. According to Pelletier et al. (2003), for example, a network is considered to be in good condition when $\lambda \leq 0.2$, acceptable when $0.2 < \lambda < 0.4$, and bad when $\lambda \geq 0.4$ (expressing λ in metric units).

To calculate reliability, the failure rate λ can be evaluated by means of regression formulas that link it to pipe diameter, D (Su et al. 1987; Mays 1989). However, it is necessary to be careful when evaluating λ by using expressions given in the literature because of the many uncertainties related to local situations. Furthermore, it is important to underline that, when evaluating the reliability of a network with respect to mechanical failures, only repairs that require a pipe to be out of service are to be taken into consideration; generally, these repairs account for 30–60% of all repairs (Ciaponi 2009).

Regarding Mean Time to Repair, MTTR, Eqs. (12) and (14) show that failure probability of a pipe is directly proportional to its unavailability U, which, in turn, depends almost linearly on its MTTR. Therefore, MTTR evaluation is notably important when evaluating reliability in connection with mechanical failures, as shown by Walters and Knezevic (1988). MTTR data is quite scarce, so that it is generally assessed by using indicative values. The most commonly used value is 1 day (Khomsi et al. 1996; Gargano and Pianese 2000; Shinstine and Lansley 2002).

Hydraulic Simulation

To correctly calculate reliability indices according to the procedure previously described, it is necessary to evaluate the actual flows delivered when the pressure at one or more nodes is less than that needed to fully satisfy demand. The proposed procedure is based on a Pressure Driven Analysis (PDA) approach giving a solution that satisfies not only the mass and energy balance equations but also the $Q_j = f(H_j)$ equations, which link, node by node, delivered flow to available pressure. This presents two rather complex problems.

The first problem concerns the definition, for each demand node, of the link between delivered flow Q_j and pressure h_j (in turn related to total head H_j). This link depends on many factors among which, of particular importance, are configuration and dimensions of the secondary network fed from the node and the spatial distribution (planimetric and altimetric) of the delivery devices. Evaluation of these factors, which may vary greatly for different nodes, requires detailed information that is practically never available for normal diagnostic activities. Consequently, $Q_j = f(H_j)$ can be applied to hydraulic analysis only by using general, approximated schemes.

In general, the flow actually delivered is expressed as a function of the required flow Qr_j, by the following relationship (Gupta and Bhave 1996):

$$Q_j = \alpha_j Qr_j \tag{15}$$

where $0 \leq \alpha_j \leq 1$ depending on pressure.

The most diffused scheme sets two-threshold values for the nodal head:

- $H\min_j$ = nodal head value below which delivery is zero; this value can be assumed to be equal to the z_j ground level or equal to $2 \div 3$ m [$6 \div 17$ in] above it, and
- Hr_j = head value required to meet demand Qr_j (m or in)

and assumes the following relationships:

$$\alpha_j = 1 \qquad \text{for } H_j \geq Hr_j \tag{16}$$

$$\alpha_j = 0 \qquad \text{for } H_j \leq H\min_j \tag{17}$$

$$0 < \alpha_j < 1 \qquad \text{for } H\min_j < H_j < Hr_j \tag{18}$$

To define α_j values according to Eq. (18), the literature proposes different expressions (Wagner et al. 1988; Fujiwara and Ganesharajah 1993; Fujiwara and Li 1998; Tucciarelli et al. 1999) that may give very different evaluations without any rational criterion to support the choice.

The procedure presented in this work uses the Wagner et al. (1988) relationship as follows:

$$\alpha = \left(\frac{H_j - H\min_j}{Hr_j - H\min_j}\right)^{1/\beta} \tag{19}$$

where β typically assumes a value of 2.

Another problem that arises regards the numerical solutions involved in the PDA approach.

According to the formulations recently proposed in the literature (Todini 2003; Cheung et al. 2005), it seems that the primary approach is to solve the entire system of mass and energy balance equations, integrated with equations representing the relationship between delivered flow and pressure at the node. Recently, some authors (Giustolisi et al. 2008; Wu et al. 2009; Giustolisi and Laucelli 2011) proposed complex and evolved methodologies for pressure-dependant demand calculation, principally on the basis of an extended global gradient algorithm (EGGA). Nonetheless, at present, the numerical methods for system solution have not been sufficiently tested, especially regarding the aspects related to their convergence, which could be problematic because of the structure of the relationship $Q_j = f(H_j)$. Because of these problems, the choice made in the paper was to model the pressure-dependant demand as suggested by some authors that have adopted calculation procedures which, although formulated according to the PDA approach, use the conventional DDA hydraulic solvers.

One of the most interesting of these procedures is that based on an EPANET-2 option (Reddy and Elango 1989; Rossman 2000), which allows water delivered through particular devices (emitters) to be modeled considering that the outward flow is linked to the nodal head by the following relationship:

$$Q_j = C_j\,(H_j - z_j)^{\gamma} \tag{20}$$

where $\gamma = 0.5$.

Assuming that in Eq. (20) the delivered flow Q_j is equal to that required Qr_j when H_j is equal to Hr_j the following is obtained:

$$C_j = \frac{Qr_j}{(Hr_j - z_j)^{\gamma}} \tag{21}$$

which, substituted in Eq. (20), leads to a $Q_j(H_j)$ expression which, for $H\min_j = z_j$ and for $\gamma = 1/\beta$, is equivalent to Eq. (19) when $0 < H_j < Hr_j$. Because Eq. (19), unlike Eq. (20), is limited to the range $0 < H_j < Hr_j$, it is necessary to adopt an iterative procedure involving the following steps:

1. A DDA conventional calculation,
2. A second calculation with emitters at $H_j < Hr_j$ nodes, using the C_j value calculated in Eq. (21), and
3. A third calculation assuming $Q_j = 0$ at the emitter-nodes where $Q_j < 0$ and $Q_j = Qr_j$ at the emitter-nodes where $Q_j > Qr_j$.

Further Considerations regarding the Procedure

As explained previously, the proposed methodology consists of a sequence of hydraulic analyses of the WDS taking into account the different working conditions (each obtained excluding a pipe from the network). For each working condition, analysis considers the n demand value obtained for each node, modulating the mean delivered flow according to hourly coefficients.

The number of working states (each characterized by unavailability of a pipe) is clearly equal to the number of pipes (NT). The state corresponding to full network efficiency (all pipes working) has then to be added to identify any problems in the network when it is operating in normal conditions. Hence, the total number of working states of a system is

$$\mathrm{NS} = \mathrm{NT} + 1 \tag{22}$$

Because, for each working state, the network has to be hydraulically analyzed for the n demand values, the number of hydraulic analysis calculations NC is

$$\mathrm{NC} = n\mathrm{NS} \tag{23}$$

As already mentioned, hydraulic analysis applies the PDA approach by using EPANET-2 with the aforementioned emitter option. Subsequently, having obtained the values of flows actually delivered (Q_j), local indicators Eq. (2) and global indicators Eq. (3) can be calculated for each working state. These indices are then used to calculate indicators Eqs. (4) and (5) taking into account the probability w_k of each working state. For this, w_k values need to be calculated by way of Eqs. (13) and (14), which require the failure rate λ and the MTTR to be assigned for each pipe.

Finally, after having calculated the corrective coefficients Eqs. (6) and (7), the global index Eq. (5) can be corrected by using Eq. (8).

Certain specifications are opportune, especially for demand modeling.

The aforementioned procedure takes into account demand variability by way of a single pattern, which represents the hourly/two-hourly variability in a typical day. This allows, as demonstrated previously, the number of simulations for each working condition to be limited to 24 (or 12 in the case of a two-hourly pattern). It is not possible, however, to take into account the variation in demand over the course of a year. That said, the proposed procedure allows one to define different demand patterns for different periods in a year. For example, the performance analysis of a network can be carried out by using two demand patterns, one representing particularly high demand levels typical of the summer period (4 months) and the other representing lower demand levels occurring during the rest of the year. Clearly, if two or more demand patterns are used, the indices in this paper have to be suitably redefined.

It must be highlighted that the proposed procedure aims to evaluate the mechanical reliability of the network according to the ordinary demand conditions, not exceptional ones such as when large flows of water are needed for firefighting. Obviously, assigning the flows needed for firefighting, it is possible to

Table 1. Supply Nodes Data

		Solution A		Solution B	
Node	z (m)	H (m)	Q (l/s)	H (m)	Q (l/s)
116	80	125.00	350.00	125.00	960.50
139	65	123.00	300.00	unknown	0.00
143	80	121.00	310.50	unknown	0.00

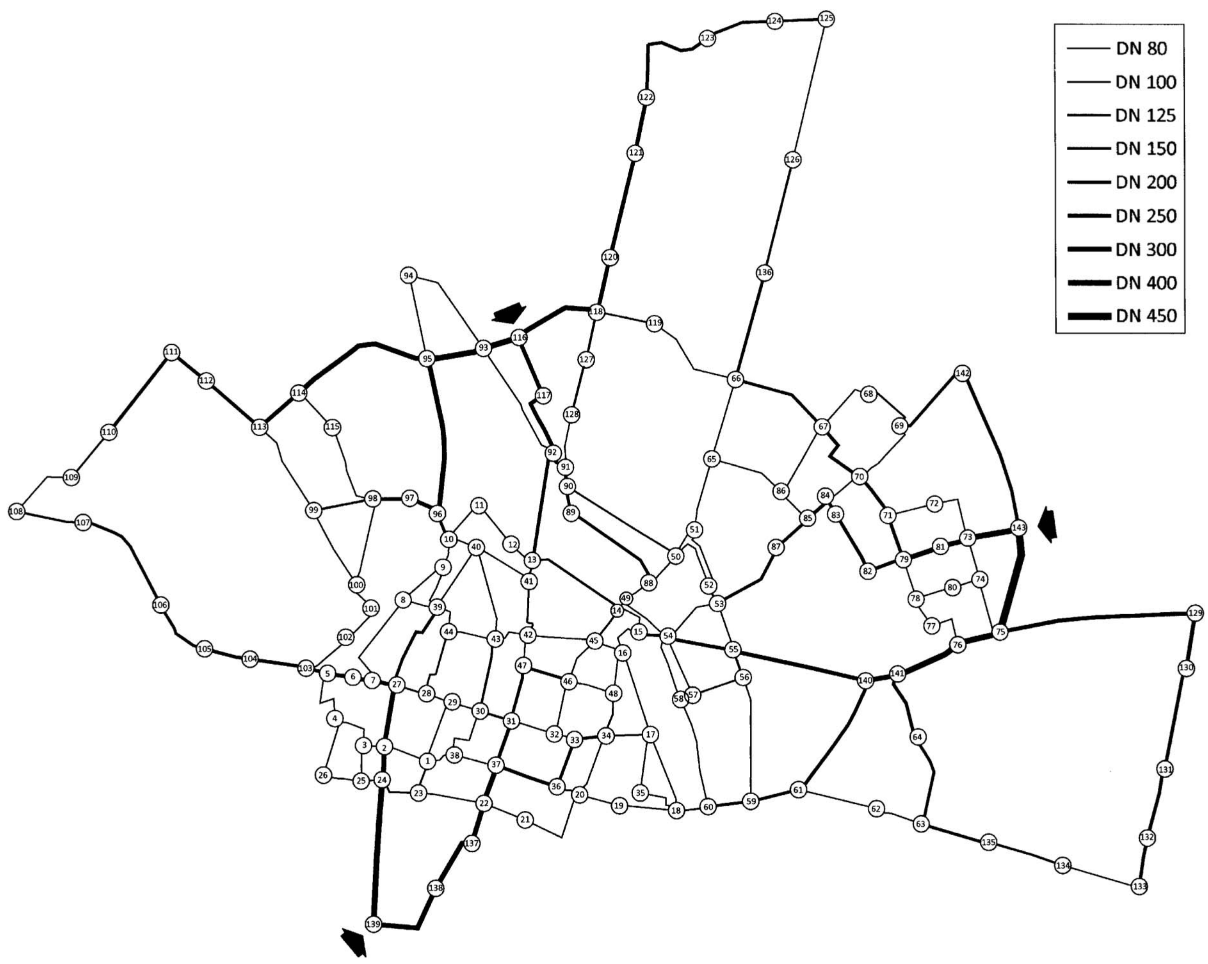

Fig. 1. Scheme of Pavia water distribution network (solution A)

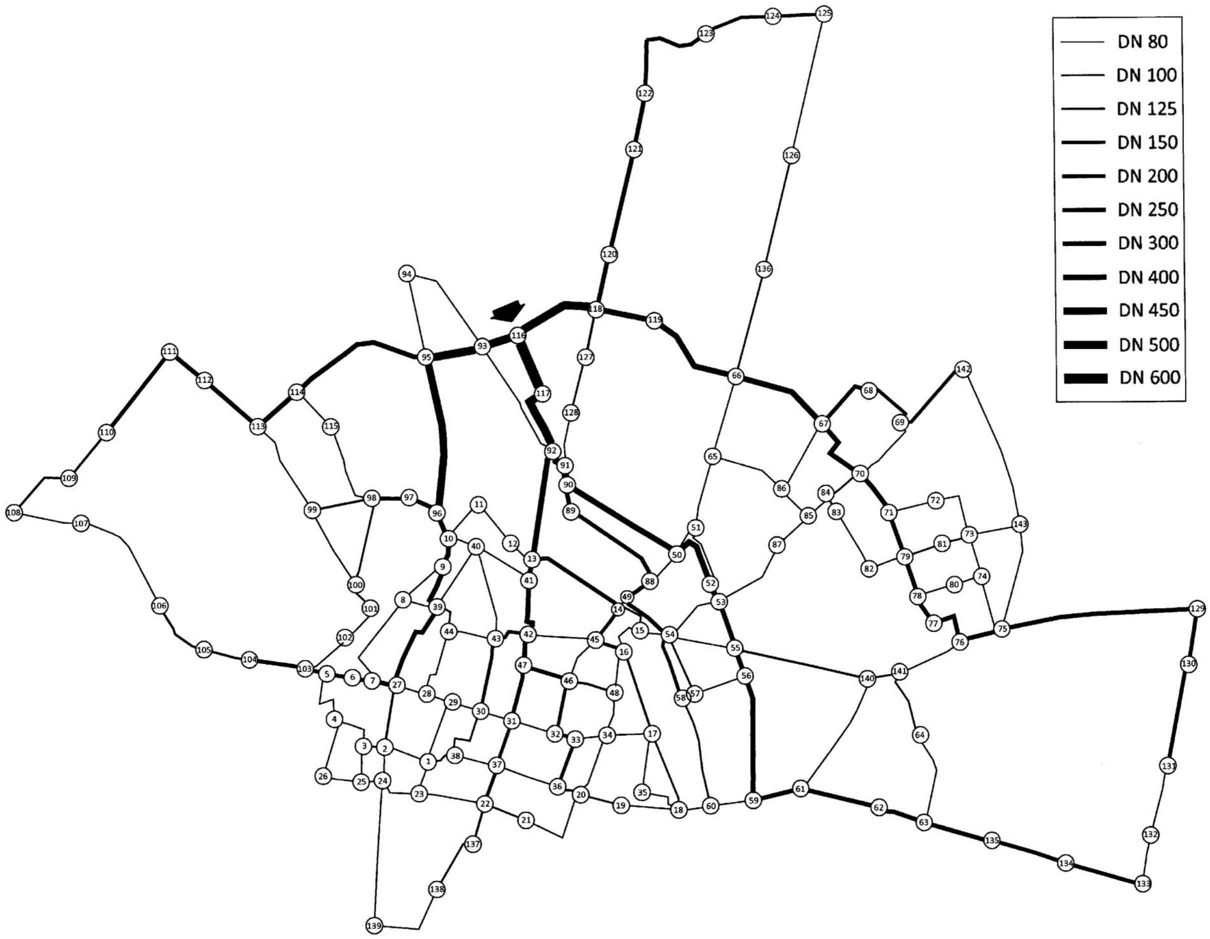

Fig. 2. Scheme of Pavia water distribution network (solution B)

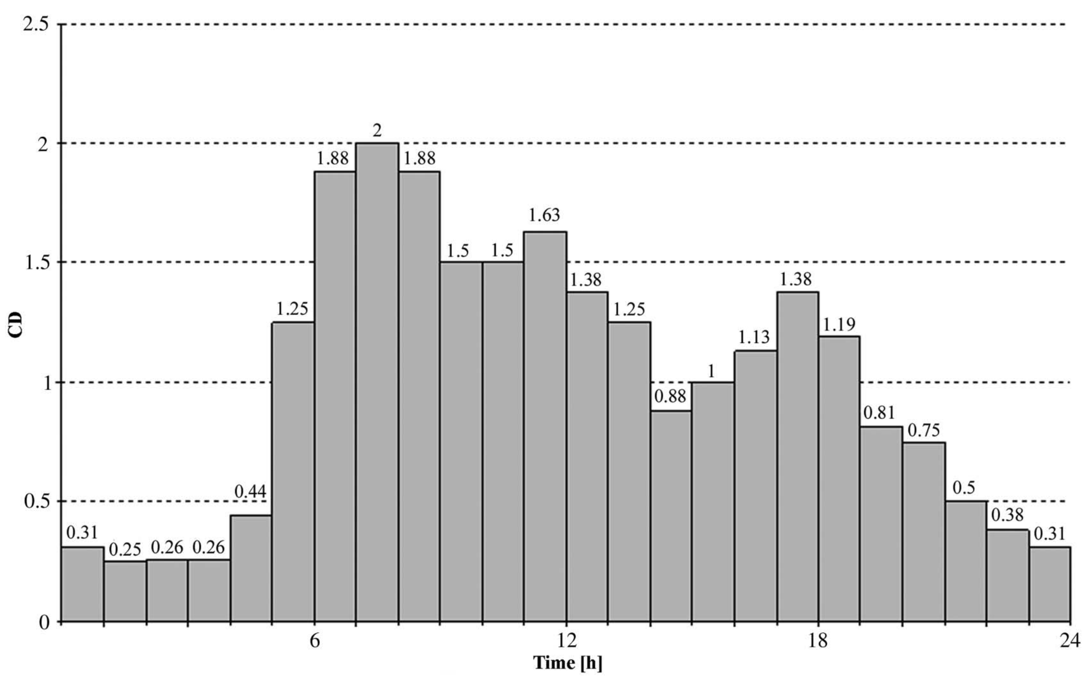

Fig. 3. Example of demand pattern

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calculate the performance indicators relevant to this extraordinary working condition. However, because it is difficult to establish the probability of a fire occurring, such an analysis has to be distinguished from that carried out by using the probabilistic approach, which refers to the unavailability of a pipe.

Procedure Application

The aforementioned procedure has been tested by comparing two different solutions for the Pavia (Italy) WDS; the network skeletonized scheme, presented in Fig. 1, is made up of 143 nodes, 206 pipes, and 64 close circuits.

In the first solution (solution A), the network has 3 water supply points (nodes 116, 139, 143); in the second (solution B), only one water supply point (node 116) exists as shown in Table 1.

In solution B, nodes 139 and 143 are demand nodes with $Qr = 0$.

In both cases, the network was designed by using a linear scheduling procedure, which divides the minimum cost design problem into two subsequent calculation phases, both of them solvable by linear programming; the first phase involves calculating the design pipe flows that minimize the water path; the second phase involves calculating the pipe diameter which minimize costs (Ciaponi and Papiri 1985). The design calculations are based on peak flows (hourly peak coefficient is 3), assuming a minimum pressure of 3.5 bars at each node. Considering that, for full functionality, 1 bar is necessary above the roof gutter and because in Pavia many buildings are present with 6–7 floors [approximately 20 m (56 inches)], 3.5 bars at the nodes of the public network is typical.

Networks consist of steel pipes lined with bitumen (Hazen-William C coefficient equal to 140).

In the next Fig. 1 (representing solution A) and Fig. 2 (representing solution B), networks are drawn by using different thicknesses that represent the different diameters of pipes.

Geometric data of the network and mean flows delivered from each node, and design results (both of case A and case B) are not given in this report for reasons of space but they can be found at the following link: http://www-3.unipv.it/webidra/materialeDidattico/ciaponi/Water_distribution_network_of_Pavia.pdf.

It is obvious, without doing calculations, that solution A is more reliable (qualitative assessment). However, to evaluate the costs and benefits of both solutions in the most rational way, it is also necessary to make a quantitative assessment even if it is incomplete and approximate. This is the reason why the proposed method is useful.

The procedure has been applied to solutions A and B assuming the single demand pattern shown in Fig. 3.

For the probabilistic evaluation of pipe failure, $\lambda = 0.2$ breaks/(km year) [0.32 breaks/(mile year)] (constant for the entire network) and MTTR = 1 day have been assumed.

Performance analysis is based on calculation, for both solution A and B, of the indicators RR and RR_c; in Figs. 4 and 5 these indices include A and B as subscripts to distinguish the different design solutions.

To improve the quantitative elements useful for comparing the two solutions, a sensitivity analysis regarding an increase in pipe roughness and in water demand has been carried out. Regarding pipe roughness, the calculations have been repeated by using head-loss coefficients (K) increasing from 1 to 5. For the K values between 1 and 5, the Hazen-Williams C coefficient is between 140 and 58.6, as shown by Eq. (24).

$$\frac{C}{C_0} = \frac{1}{K^{\frac{1}{1.85}}} \tag{24}$$

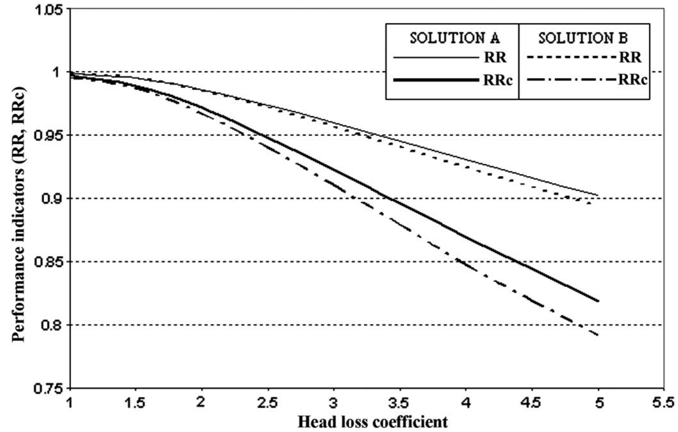

Fig. 4. Trend of performance indicators depending on head-loss coefficients

in which C_0 = design value of Hazen-Williams coefficient (140); C = current value, related to K; and K is the head-loss multiplier.

The trend of the performance indicators, in function of head-losses coefficients (K), is shown in Fig. 4.

Regarding the increasing water demand, the calculations have been repeated by using demand coefficients between 1 to 2. The trend, depending on demand, is shown in Fig. 5.

Looking at Figs. 4 and 5, it is clear that solution A is more reliable than solution B. The proposed method, however, reliability assessment quantitatively, not just qualitatively.

In particular, the analysis shows that both solutions are very reliable (reliability almost equal to 1) when water demand and pipe roughness are in-line with design specifications. However, when the levels of pipe roughness and water demand are different than those planned, solution A is much more reliable than B.

Furthermore, the difference between RR and RR_c shows that both the spatial distribution of water deficits and their duration affect the reliability of the system. This effect is more significant with solution B, which renders the network less able to function satisfactorily in critical working conditions.

In addition, it is important to highlight that the reliability values (RR and RR_c) depend on how much detail is used to describe water distribution systems and the more detail used, the higher RR and

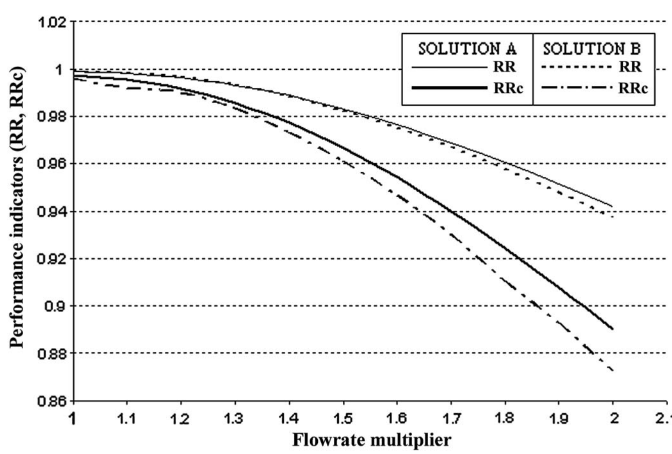

Fig. 5. Trend of performance indicators depending on demand coefficients

RR_c. This is because the more important the pipe isolated, the greater the hydraulic disturbance.

The proposed method is, therefore, sensitive to the degree of network skeletonization and, when skeletonization is high, the level of reliability is underestimated.

It follows that reliability assessment must be carried out by using a detailed description of the system but this is valid practice for any procedure used. In fact, the more detail used to describe the network, the closer the reliability assessment is to the real system performance.

The network example given in this report is quite skeletonized. Therefore, the reliability values obtained are underestimated. However, the studied case is purely an example. This aside, even though the performance values obtained in skeletonized cases are not valid in the absolute sense, they are useful for comparing different solutions.

Conclusions

The method proposed in this paper allows for the assessment of the reliability of a WDS, taking into account the probability values of the delivery deficits caused by the various cases of mechanical failure, which can occur during a system's operative life.

The work presented in the paper takes exclusively into account the unavailability of pipes.

The procedure may be adapted to include also pumps unavailability; to do this, it is simply necessary to add the hydraulic analysis and the reliability indices calculation for the correspondent scenarios; the real difficulty consists of the definition of probability to be associated with these scenarios.

The reliability value is given by simple overall indicators, but the procedure also provides local performance indicators, defined for each node and for each working condition. Analysis of local indicators allows the identification of the network zones that are more affected by disservices. For these situations, a detailed analysis of hydraulic simulation results permits the identification of the weak links in the system and to address or parallel them.

The procedure has certain limitations. First, the method is based on the assumption that each single pipe can be isolated (for maintenance or repair). In reality, the valves are not necessarily at the end of each pipe. Isolating one pipe, therefore, often involves isolating others. However, the method can be adapted to consider a group of pipes (a segment) as a unit for reliability assessment once the group has been identified by using methods recently published. Linked to this is also the fact that the relevance of the isolated pipe and the subsequent effect on hydraulic performance depends on how skeletonised the network is.

It follows that the reliability evaluation requires a detailed description of the system, as is the case for any procedure one may use. In fact, the more detail used to describe the network, the closer the calculated reliability value is to the actual value.

Another limitation is the fact that water demand is established with reference to an average day.

Also this problem can be overcome by using more demand patterns, each corresponding to a particular interval in time (e.g., a season).

Despite these limitations, the proposed method can be a useful decision tool for designing a new WDS or for improving existing networks, especially when the objective is to compare different solutions by using a rational criterion.

The method can also be easily applied, as shown in the example, to evaluate system reliability with reference to variations in design parameters, such as an increase in water demand and/or a drop in hydraulic conductance resulting from pipe lining degradations (corrosion or deposits).

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