Field Data–Based Methodology for Estimating the Expected Pipe Break Rates of Water Distribution Systems

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Abstract: Presented in this paper is a field data–based probabilistic approach to quantifying the expected pipe break rates of water distribution systems. Uncertain demands and variations in the roughness of pipes during their service lives are described as random variables. Sample values of these random variables are generated and input to a distribution system model to determine the resulting minimum and maximum pressures in Monte Carlo simulations. Based on an estimated break rate–maximum pressure relationship, the sample maximum pressures obtained from a Monte Carlo simulation are transformed into a sample of break rates, and the expected pipe break rate can subsequently be determined. The sample minimum pressures are used to gain a better understanding of the distribution network. This probabilistic approach is used for a part of the City of Hamilton's network in Ontario, Canada. The results show that the frequency of low-pressure events is very small, but a higher minimum pressure criterion would inevitably increase expected pipe break rates. Local field data collection is necessary to use the proposed methodology. Savings resulting from reduced pipe break rates justify costs associated with data collection.

DOI: 10.1061/(ASCE)WR.1943-5452.0000686.

Author keywords: Water distribution systems; Probabilistic; Pipe breaks; Design.

Introduction

The aim of a water distribution system (WDS) is to safely deliver water to all customers of the system in sufficient quantity and quality as economically as possible. To ensure safe and reliable delivery of water across a WDS, system pressure should generally be maintained between minimum and maximum acceptable levels. Pressure is a key factor in operating WDSs and must be carefully managed. Its excess or deficit can cause hazard or inconvenience. In standard WDS design, it should be ensured that all pressures throughout the system are above a minimum pressure, known as the minimum pressure criterion (MPC), when the system experiences worst-case loading, which is considered to be the greater of the maximum hour demand and the maximum day demand plus fire flow (Filion et al. 2007b). The MPC is mainly established to prevent direct contamination and to provide safe drinking water from the source to all individual taps.

For safe, reliable, and economic WDS operation, various local standards for pressure have been established so that sufficient pressure is always provided (but not so high as to cause danger). Although inadequate pressure and lack of pressure monitoring are public health concerns, excessive pressure is seldom a regulatory criterion but can also be problematic (LeChevallier et al. 2014). Creaco et al. (2016) provided a methodology for energy and leakage minimization in which the relationship between demand and service pressure and the relationship between leakage and service pressure were first assessed. Next pumping energy consumption was optimized based on the on-off setting for pumps (pump settings were expressed as a function of water level in the tank) associated with different service pressure values. The energy and cost savings associated with pump operation were then assessed. Finally the manner in which variations in district service pressure affecting leakage, electricity costs for the pump operation, and pipe break rates was assessed. An average service pressure reduction from 48.23 to 30 m (i.e., a 38% reduction in delivery pressure) in the Abbiategrasso district, Italy, led to reductions of 27% in leakage and 5.3% in pipe break rates, and energy savings of 53%. It is clear that excess pressure can cause higher burst rates, increased leakage, and higher costs. There is a direct link between system pressure and pipe break rates.

Pipe breaks are inevitable and cause significant water losses. An average of 850 water main breaks occur daily in North America, with a total annual repair cost of more than $3 billion (http://www.watermainbreakclock.com). Frequent occurrence of water main breaks is a concern of municipal decision makers worldwide. Pipe failures are commonly classified into two main categories: leaks and bursts. Leakage losses that have a flow rate below a certain threshold value are categorized as background leakage (Lambert and Hirner 2000). Water utilities can use leakage as a metric to evaluate the condition of the water system given that more leakage is often associated with deterioration in the system's physical condition (Lambert et al. 2013). Thus, reducing leakage and replacing leaky pipes can help reduce pipe break rates. Thornton and Lambert (2006) and Lambert et al. (2013) developed practical prediction methods and empirical equations to estimate the beneficial influences of pressure management on leakage and burst frequency. Water utilities can assess leakage using leak detection and location techniques. These techniques enable water utilities to develop performance indicators to assess water losses and benchmark themselves with other water utilities (Fanner et al. 2007; AWWA 2009; Hughes et al. 2011).

Only a few studies, using limited field data, have investigated the relationship between pressure and pipe breaks (Lambert et al. 2013; Martınez-Codina et al. 2015). Field data are limited because

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Note. This manuscript was submitted on December 31, 2015; approved on March 23, 2016; published online on June 13, 2016. Discussion period open until November 13, 2016; separate discussions must be submitted for individual papers. This paper is part of the ***Journal of Water Resources Planning and Management***, © ASCE, ISSN 0733-9496.

of a historical lack of awareness of its importance. Indeed, a high operating pressure can result in more frequent pipe breaks. Although the WDS design strategy ensures that pressures in the system during peak demand periods are not less than a minimum, high pressures during low-demand periods—late night and early morning—and at low elevation areas may cause pipe breaks (Thornton and Lambert 2006; Fanner et al. 2007), but this does not mean that most failures occur during minimum or night flow conditions. Pressure management is traditionally regarded as an effective way to control leaks and pipe bursts during off-peak hours (Gomes et al. 2011). Several studies have been conducted to determine flow control valve settings to minimize leakage losses (Creaco and Pezzinga 2014). As part of pressure management, minimum night flow analysis is conducted to identify factors affecting losses because most users are not active during the night and pressures are high throughout the system (Walski et al. 2006a; Campisano et al. 2012). In pressure management, however, optimizing the number of valves and their locations and determining the optimal adjustment of valve openings are all challenging tasks. The problem of optimal location and regulation of control valves for leakage reduction and excess pressure minimization has been widely investigated (e.g., Jowitt and Xu 1990; Ali 2014; Creaco and Pezzinga 2014). Reducing pressure using pressure-reducing valves (PRVs) might result in only minor energy savings (LeChevallier et al. 2014). Pressure management is frequently considered in system master planning, engineering studies, or hydraulic modeling. Practitioners can better incorporate optimization for pressure management if they have a clear understanding of the relationships between pressure and other distribution system performance indicators, such as leakage, breaks, and energy usage (LeChevallier et al. 2014). No effort has been made to develop an indicator so that pipe break rates can be incorporated into the design of WDSs for long-term economic efficiency.

Distribution of pressures in water networks depends on pumping heads and water levels in tanks, pipe diameters and roughnesses, and water demands. Because the numbers and types of future consumers cannot be exactly determined, the future demands for WDS design are uncertain. The roughness coefficients of pipes are also uncertain because of aging during the period of operation. Consequently, computed pressure, an important factor in WDS planning and design, is not certain, either. Therefore, to gain a clear picture of how system pressure affects the potential for pipe breaks, demands and hydraulic conductivity of pipes should be treated as uncertain quantities and modeled with probability distribution functions (PDFs).

To address uncertainty in WDS design, several models have been developed to improve WDS performance at minimum cost (Lansey et al. 1989; Kang et al. 2009; Filion et al. 2007b). Risk-based optimization has been used to incorporate uncertainty in the design of WDSs (Tung 1986; Xu and Goulter 1999; Kapelan et al. 2006; Yannopoulos and Spiliotis 2013). In many proposed methods for determining optimal WDS design , uncertainty is usually incorporated into the problem formulation as a constraint to either maximize overall robustness (i.e., the probability of satisfying minimum pressure constraints at all network nodes) or to minimize total WDS risk (i.e., the probability of pressure failure at any nodes). Reliability analysis has been conducted for better WDS design. In reliability-based optimal design , reliability indices are generally incorporated into the optimization framework as a constraint in order to maximize overall system reliability (Babayan et al. 2005; Kapelan et al. 2005; Gomes and Karney 2005; Atkinson et al. 2014). However, these studies have focused primarily on the hydraulic performance of systems in an attempt to meet basic delivery requirements.

In chance-constrained optimization schemes for WDS design, pipe breaks are modeled stochastically (Shinstine et al. 2002; Filion et al. 2007b). Pipe break rates are often used as an index of system performance by practitioners, but little effort has been made to develop methods to better quantify mean pipe break rates. Mean pipe break rates determined by considering a wide range of uncertain demands and pipe roughnesses may be used as an indicator in design and to help redefine what is optimal.

This study explores the linkage between WDS maximum operating pressure and expected (or mean) pipe break rates. At a practical level, it provides a more comprehensive understanding of the effect of system pressure and pressure standards on pipe break rates. Quantified mean or expected pipe break rates serve as an indicator for designers (e.g., it can be incorporated into optimization models as an additional objective function in order to minimize expected pipe break rates), and can help a utility to strike a balance between pipe breaks and cost . Uncertain water demands and pipe roughnesses are modeled with a Monte Carlo simulation (MCS) algorithm, which is used for computing expected daily break rates over an extended period. This probabilistic approach is applied to a portion of the City of Hamilton, Ontario's water network to determine expected yearly break rates.

This paper comprises three parts. In the first part, prediction models and causes of pipe breaks are briefly reviewed. In the second part, a probabilistic approach for computing expected break rates is presented. In the last part, the probabilistic approach is applied to a case study to quantify pipe break frequency. Also, the effect of MPC on system pressure and pipe break rates is examined.

Prediction Models for Pipe Breaks

Several models for predicting water main break rates have been developed to show break behavior and break patterns. These models can be classified into deterministic and probabilistic categories (Kleiner and Rajani 2001). Deterministic models often use two- or three-parameter equations to derive breakage patterns based on pipe age and diameter and breakage history (Kleiner and Rajani 2001). The division of pipes into groups with similar properties (operational, environmental, and pipe type) is often necessary, requiring efficient grouping schemes (Makar and Kleiner 2000). Probabilistic models are used to estimate pipe life expectancy or failure probability. The efficiency of rehabilitation planning according to projected pipe break patterns depends on the quality and quantity of available data (Kleiner and Rajani 2001).

Shamir and Howard (1979) and Walski and Pelliccia (1982) used exponential functions to predict break rates based on recorded break data. Clark et al. (1982) analyzed replacement costs to determine the optimal timing of pipe replacement. Kettler and Goulter (1985) suggested a linear relationship between pipe breaks and age based on a sample of pipes installed within a 10-year period in Winnipeg, Manitoba. Jacobs and Karney (1994) proposed a model using GIS to estimate the probability of occurrence of a day with no pipe breaks and the probability of an independent pipe break, which they defined as a break that occurs more than 90 days after and/or more than 20 m from a previous break. Kleiner and Rajani (2001) reported that the breakage rate for buried pipes could be related to pipe deterioration, climatic conditions, and soil shrinkage behavior. Achim et al. (2007) developed a neural network model to predict the number of breaks/km/year for water mains based on three years of recorded data in Melbourne, Australia. Wang et al. (2009) developed five deterioration models that predict annual water main break rates considering pipe material, diameter, age, and length.

Le Gauffre et al. (2010), using available data for 1993–2010 for the Greater Lyon area in France, derived relationships between pipe break rates and climate variables, including the number of hours with temperatures lower than 0°C, the number of hours with temperatures higher than 30°C, the maximum number of consecutive days with daily precipitation less than 1 mm, and the maximum number of consecutive days with daily precipitation greater than 1 mm. They concluded that rainfall and freezing duration tend to increase pipe break rates. Kimutai et al. (2015) studied three statistical models—the Weibull proportional hazard model (WPHM), the Cox proportional hazard model (Cox-PHM), and the Poisson model (PM)—for predicting pipe failures in the City of Calgary's water network. Their results indicated that WHPM and PM are suitable for metallic and PVC pipes, respectively. From the statistical models, they also showed that physical covariates (e.g., pipe diameter, length), when compared with environmental covariates (e.g., temperature), are more critical in affecting pipe failure rates.

Previous studies relied on field data and did not focus on the relationship between pressure and pipe breaks.

Causes of Pipe Breaks

Many factors contribute to the deterioration of pipes that eventually result in pipe breaks. These factors include pipe age, water pressure, temperature, soil corrosivity, water contents of surrounding soils, previous pipe breaks, pipe diameter, pipe material, and construction practices (Wang et al. 2009; Morris 1967; Kleiner and Rajani 2001). A study of the New York water supply system conducted by the U.S. Army Corps of Engineers (USACE 1981) revealed that leakage increases the moisture content of the surrounding soils and expedites corrosion. Some studies indicated that more breaks are expected in a water network as temperature decreases (O'Day 1982; Kimutai et al. 2015). Each factor's relative weight of contribution to pipe breaks is still not universally agreed on in the literature, but it has been established that some factors carry more weight than others. It was identified that the majority of breaks occur in cast iron (CI) pipes, which are the oldest pipes, often installed more than 50 years ago (Pelletier et al. 2003; Singh and Adachi 2013; Kimutai et al. 2015).

Pipe diameter was identified as one of the factors affecting pipe failure rates (Clark et al. 1982; Berardi et al. 2008). It has been consistently reported in the literature that small-diameter pipes have a greater number of failures than do large-diameter pipes (Berardi et al 2008; Wang et al. 2009; Kimutai et al. 2015). The majority of statistical models used for predicting water main breaks consider pipe age as an important factor (Berardi et al. 2008; Wang et al. 2009). It has been reported by many researchers that pipe failure varies with pipe age in accordance with a bathtub curve (Andreou et al. 1987; Kleiner and Rajani 2001; Singh and Adachi 2013). Environmental factors such as precipitation, soil conditions, frost and traffic loading, and the quality of external groundwater have been identified as factors contributing to the failure rate of pipes in water networks (O'Day 1989; Rajani and Zhan 1996; Kleiner and Rajani 2001; Kimutai et al. 2015). Generally, low temperature and rainfall tend to increase pipe break rates (O'Day 1982; Brander 2001; Kimutai et al. 2015).

As explained previously, many factors influence pipe breaks (e.g., pipe deterioration, pipe properties, and environmental conditions). Without pressure, however, water would only be nominally present in a distribution system (no leak and no pipe breaks would occur) and the system would be unable to deliver water to users. Thus, pipe breaks mostly occur because of pressure. Few studies have been conducted to demonstrate the effects of pressure management on reducing pipe break rates. Pearson et al. (2005) illustrated that reducing pressure by approximately 50 m during high-pressure events with the installation of control valves in a real system, part of a large network in the United Kingdom, causes a dramatic reduction in burst frequency. The researchers showed that burst frequency dropped from 3 per month to 1 every 6 months on average. They also established a relationship between relative burst frequency and pressure based on collected data from 50 WDSs:

$$\frac{B_1}{B_0} = \left[\frac{(P_1 - P_a)}{(P_0 - P_a)}\right]^{N_2} \tag{1}$$

where B_0 and B_1 = burst frequency before and after pressure reduction, respectively; P_0 and P_1 = average pressure before and after pressure reduction, respectively; P_a = pressure below which there are no breaks (P_a was suggested to be 20 m); and N_2 = exponent with mean values ranging 1.24–1.32.

Thornton and Lambert (2006) analyzed data collected from 21 utilities in 11 countries on breaks (or repairs) before and after pressure management. They concluded that the percentage reduction in pipe bursts often exceeds the percentage reduction in average maximum operating pressures. For example, in Halifax, Canada, an 18% reduction in average maximum operating pressure caused a 23% reduction in pipe breaks. Lambert et al. (2013) summarized work on predicting the benefits of pressure management in WDSs and concluded that reducing excess pressure could have a substantial influence on reducing bursts (e.g., a 1% reduction in average pressure would cause a 1.4% reduction in burst frequency). They also developed a relationship between burst frequency and pressure from data collected by an Australian utility:

$$BF = BF_{\mathrm{npd}} + A \times (\mathrm{AZP}_{\max})^{N_3} \tag{2}$$

where BF = burst frequency; BF_{npd} = pressure-independent burst frequency; $\mathrm{AZP}_{\max}$ = maximum pressure at the average zone point (AZP); A = coefficient influencing the slope of the pressure-dependent part of the relationship; and N_3 = exponent recommended to be close to 3.

The value of BF_{npd} can be estimated as the lower boundary of the data points in a plot depicting burst frequency as a function of average zone night pressure (Lambert et al. 2013). Martınez-Codina et al. (2015) presented a new methodology based on a maximum pressure indicator for identifying the range of maximum pressures that would most likely reduce pipe breaks. They concluded that the maximum pressure should have an upper limit to reduce the probability of breaks (e.g., the upper limits of 79, 96, and 70 m for three case studies in Madrid, Spain, were presented). However, they suggested that as pipes age and deteriorate, these thresholds need to be updated and that model results may change with time.

Expected Pipe Break Rates

High pressures coupled with the physical and environmental conditions of pipe networks result in an increase in water main breaks. This increase causes increases in operation and maintenance costs, in water loss, and in social costs such as loss of service and disruptions in traffic, business and industrial processes, and residential activities. Annual break rates (breaks/km/year) are often used as a controlling criterion in rating water main conditions. Technically, to determine the effects of parameters affecting pipe breaks—for example, pipe material deterioration, external loads (frost, traffic, and temperature), and quality of pipe installation— field data are necessary, but collection of sufficient data is a challenging task

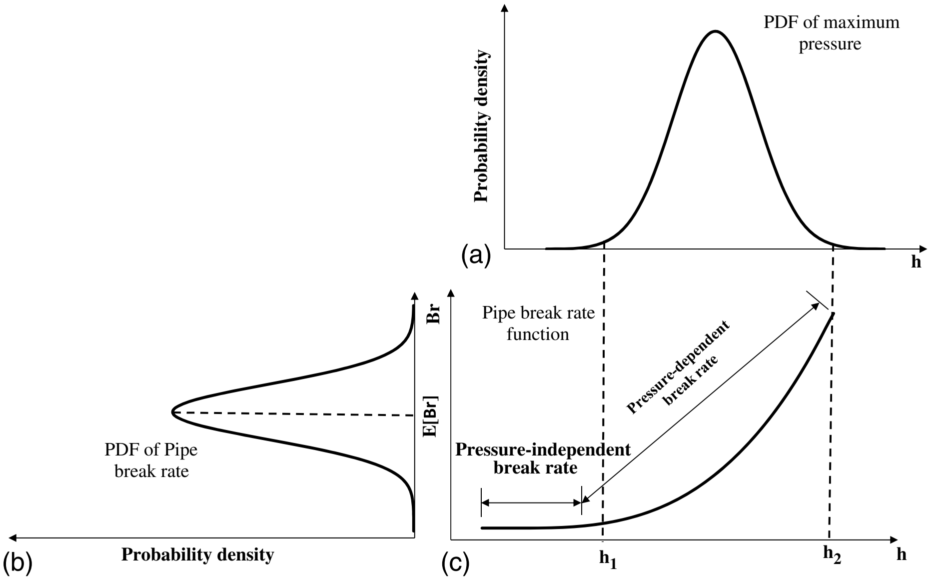

Fig. 1. PDF of pressure, continuous break rate function, and PDF of break rate

for water utilities. Sophisticated techniques must be used to sift through collected field data to identify meaningful information on the status of a distribution system (Speight 2008). Thus, break data are often segregated into homogenous groups of materials, diameters, ages, environmental and operational conditions, and mechanisms of failure to better identify breakage patterns (Kleiner and Rajani 2001; Martins et al. 2013).

The only parameter that can be easily measured in WDSs is water pressure. A relationship between pressure and pipe break rate can be extremely useful in implementing pressure management strategies and even in the design of a WDS. A pipe break rate function that maps a predictor variable at a system (i.e., maximum pressure h) to a unique average level of pipe break rate (Br) forms the basis for calculating expected pipe break rates (Fig. 1). The hypothetical brake rate function depicted in Fig. 1(b) may be valid for pipes with homogeneous properties (e.g., the same type of pipe) and under similar environmental conditions. Of course, pipe materials, the environment in which the pipes are laid, and the operating characteristics of the system also influence the likelihood of pipe breaks (Kleiner and Rajani 2001). Ideally, pipe material, size, age, type of bedding, soil characteristics, operating pressures, water temperatures, time, place, and type of historical breaks should be available to derive breakage patterns based on all factors influencing pipe bursts (Makar and Kleiner 2000). However, in many cases only partial sets of data exist. The failure rate of a distribution system can be assumed to have a relatively low value until a particular pressure is exceeded, and then the failure rate increases rapidly for small pressure increases (Lambert et al. 2013). This is as depicted in Fig. 1(b).

The brake rate curve (as depicted in Fig. 2) can move to the left over a period of years and also seasonally because of other influential parameters contributing to pipe breaks (Lambert et al. 2013). Moreover, the zone of pressure-independent failure rate may vary in WDSs because of the condition under which the pipes are laid. Other factors, such as pipe diameter, soil condition, and quality of installation, may be included in Fig. 2 to describe more accurately the relationship between pressure and break rate. Thus, for each pipe material and under each environmental and operational condition, the pipe break rate function can be represented by a unique curve. To develop such a curve, the maximum pressure indicator (the maximum pressure at the AZP of a pressure zone) is often used because it is known to be a key control parameter for reducing the number of pipe breaks in pressure management (Lambert et al. 2013; Martınez-Codina et al. 2015).

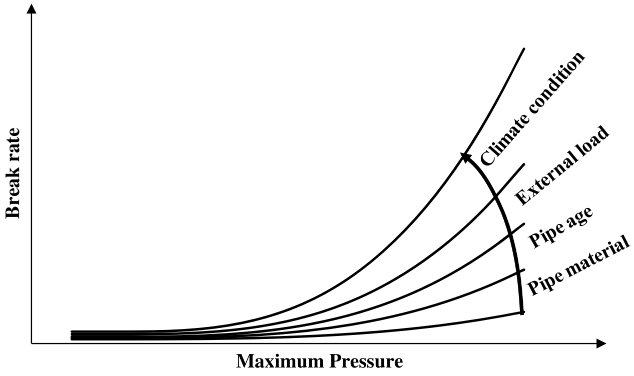

Fig. 2. Hypothetical shapes of break rate functions influenced by factors contributing to pipe breaks

Two simplifications are necessary for developing a pipe break rate function. First, no attempt is made to associate a predictor variable (i.e., maximum pressure) to a precise break rate level observed in the field during a particular period of time. In reality, if a particular predictor variable level is encountered frequently (e.g., pressures above a threshold level), break rates will vary in each instance depending on the specific circumstances at the time the system experiences those high pressures. The break rate function only associates an average break rate with each level of the predictor variable. The second simplification is that maximum pressure is assumed to be the only predictor variable controlling pipe break rates.

Once a pipe break rate function is established, an estimate of the expected pipe break rate is calculated by integrating the product of the continuous break rate function with an empirical probability density function (PDF) of the maximum pressure over the possible range of maximum pressures:

$$E[Br] = \int_{h_1}^{h_2} Br(h) f(h) dh \tag{3}$$

where $E[Br]$ = expected pipe break rate (breaks/km/day) during the service life of a water distribution system; h_1, h_2 = lower and upper limits of the maximum pressures that the system may encounter in its *lifetime* (m); $Br(h)$ = break rate function associating maximum pressure h to an average pipe break rate (breaks/km/day); and $f(h)$ = PDF of maximum pressure. Fig. 1 graphically represents this procedure. Note that the pipe break rate here is expressed as a function of random variable h—$Br(h)$. Eq. (3) theoretically gives the expected value of the pipe break rate. On this basis, numerical integration can be carried out once the break rate function $Br(h)$ and $f(h)$ are both known numerically to determine the expected pipe break rate.

The break rate function as shown in Fig. 1(b) has a logical and reasonable shape, but its exact form associated with different WDS physical and environmental conditions remains a topic for future research. The shape of a break rate function varies from one system to another and can be established using observed data. It is beyond the scope of this paper to determine the exact shape of this pipe break rate function for a specific system. However, as long as the break rate function follows a similar logical shape, the probabilistic approach presented in this paper can be used and a reasonable estimate of expected pipe break rates can be provided.

Eq. (3) provides a simple way of estimating expected pipe break rates considering wide ranges of loadings and influencing factors (e.g., uncertain demands and pipe roughnesses). The estimated expected pipe break rates can be used as an indicator for the development of economic models to determine the financial benefits of reducing pressures. In essence, Eq. (3) itself does not include all of the factors influencing pipe breaks; rather, the pipe break rate function depicted in Figs. 1(b) and 2 is used to describe the influence of a variety of factors, among them pipe characteristics, environmental conditions, and system pressures contributing to pipe breaks. In practice, pipe break rate functions can be developed based on historical pressure data obtained from supervisory control and data acquisition (SCADA) systems. However, in the absence of SCADA data for a specific system, pipe break rate functions developed for other, similar systems may be used. Limited local data may be used to verify or fine-tune pipe break rate functions developed for other, similar systems. The main objective of this study is to develop a procedure for determining an indicator that can be incorporated into the design of new networks using collected data on pipe break rates. The need for collecting adequate data on pipe breaks is also highlighted when the usefulness of the indicator is explained.

Simulation Algorithm for Expected Pipe Break Rates

Expected pipe break rates as defined previously can be computed using a MCS algorithm. Demand and pipe roughness often vary over a long time period (Bao and Mays 1990). It is assumed that a WDS during its life span experiences the entire range of possible demands and pipe roughness in order to determine system pressures under a wide range of design loads (Bao and Mays 1990; Kapelan et al. 2005). In other words, the expected pipe break rates here are computed over the range of possible high pressures considering uncertain design parameters (i.e., demand and roughness coefficient) that a WDS may experience in a typical day (one of the strongest assumptions made in the design process) to determine an additional indicator that can be used for WDS design. First, in each MCS run, one demand for each node and one roughness for each pipe are generated and entered into the network solver *EPANET2* (Rossman 2000) to compute pressure heads under a typical diurnal demand pattern. As indicated in Table 1, a MCS comprises N independent runs for M nodes of a network. The extended period simulation (EPS) should be used to ensure that pressures can stay above the MPC during peak demand times (the duration of simulation in each MCS run is chosen to be 24 h). At each run, the computed minimum pressure at each node during a 24-h simulation is compared with the MPC to ensure that nodal pressures are always above the MPC. Of course, supply pressure should be increased to ensure that the MPC is satisfied. The maximum nodal pressure in each MCS run for each node (e.g., $h_{2,1}$ in Table 1) is also recorded. These maximum pressures are used to derive the PDF of the maximum pressure. At the end of the last run, the PDF of maximum pressures is derived and the expected pipe break rate $E[B]$ is numerically computed based on Eq. (3) incorporating the associated pipe break rate function.

Table 1. Monte Carlo Simulation Runs and Calculated Maximum Pressures

	Node					
Run	1	2	3	4	. . .	M
1	$h_{1,1}{}^{a}$	$h_{1,2}$	$h_{1,3}$	$h_{1,4}$	. . .	$h_{1,M}$
2	$h_{2,1}$	$h_{2,2}$	$h_{2,3}$	$h_{2,4}$	. . .	$h_{2,M}$
3	$h_{3,1}$	$h_{3,2}$	$h_{3,3}$	$h_{3,4}$	. . .	$h_{3,M}$
. . .	. . .	. . .	. . .	. . .	. . .	. . .
N	$h_{N,1}$	$h_{N,2}$	$h_{N,3}$	$h_{N,4}$	. . .	$h_{N,M}$

[a] $h_{1,1}$ = maximum pressure observed for Node 1 during the 24-h simulation of the first MCS run. These maximum pressures were later used as a sample for estimating the PDF of maximum pressure.

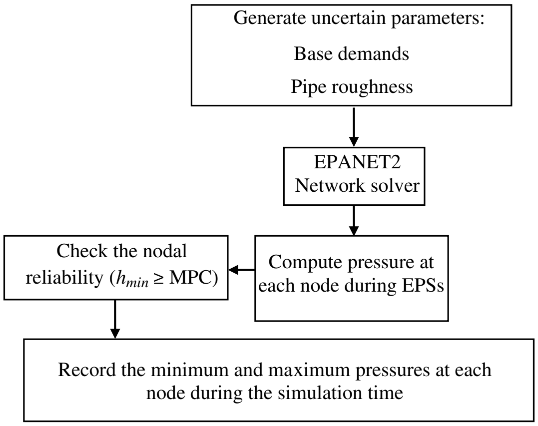

Fig. 3. Steps to compute pressures in a MCS run

The tasks performed in a MCS run are shown in Fig. 3. First, base demands and pipe roughnesses are generated according to their corresponding PDFs. The base demand usually equals the average-day demand calculated from monthly or quarterly meter readings and billing records. In WDS design, demand patterns obtained by multiplying base demand by demand multiplication factors are used to simulate the behavior of a quasi-dynamic system over a period of time during which demands and boundary conditions change with respect to time. For the proposed MCS, the PDF of each uncertain parameter can be determined on the basis of measured data of the system. However, because of the scarcity of field data, hypothetical PDFs are used for uncertain design parameters. According to the authors' review of the literature, the following limitations exist in using field data:

- In any real system, there can be hundreds or thousands of unknowns (e.g., roughness coefficient for each pipe and demand at each node) and only a relatively small number of field observations. Wu et al. (2002) observed that when the number of unknowns greatly exceeds the number of useful observations, there is little confidence in the calibration results because there are too many different parameter values that produce results close to the observed values (Walski et al. 2006b).
- Consumer demands occur along pipes at many separate locations. Nodal water demand is an aggregation of the consumption of individual houses and buildings in the vicinity and is allocated to a demand node at a junction of pipes (Kang and Lansey 2009). Even with this aggregation, because of the complexity of network systems with spatially distributed user types and lack of available field data, estimating an individual node's demand is challenging.
- Field measurements from SCADA systems play a critical role in determining nodal demands and pipe roughnesses. Given the limited number of monitoring locations, the topographical and spatial distributions of meters in networks strongly influence the quality of demand estimation (Kang and Lansey 2009). If some portions of the system are insufficiently measured, demand and roughness estimates and subsequently model predictions in those portions will be inaccurate and contain a high degree of uncertainty.
- To reduce the number of unknowns, the roughness coefficients for a subset of pipes are assigned the same value according to the pipe's age, material, diameter, and relative locations (Mallick et al. 2002), which can be achieved if the precise age of each pipe in the network is known; however, determining the year when each pipe segment was laid down can be fairly tedious or sometimes impossible, particularly for older pipes in a real water distribution system.
- Field data collection for pipe roughness estimation is generally performed using a fire flow test, which is intended to cause large head losses so as to make the system sensitive to the roughness coefficients (Ormsbee and Lingireddy 1997). Data collection for this purpose is rare, perhaps carried out every five years, because pipe roughness changes slowly.

In many studies, demands and roughness coefficients have been assumed to follow normal distributions with known means and standard deviations (Lansey et al. 1989; Gomes and Karney 2005; Kapelan et al. 2005; Filion et al. 2007a). In this paper, too, water demand and pipe roughness is assumed to be normally distributed for simplicity. In real systems, demand pairs may tend to be perfectly correlated in water networks, implying that all users react simultaneously to normal and peak demand. In this case, nodal demands are dependent of each other. To generate correlated random samples of demands, a procedure suggested by Iman and Conover (1982) can be used. The focus of this paper is not to determine the impact of correlation between demand pairs on pipe break rates. The methodology developed and presented here can certainly be used to handle correlated random variables.

In the second part of the MCS, the *EPANET2* solver is used to compute nodal pressure heads. The minimum pressure head at each node in a 24-h simulation is used to determine the nodal reliability index. Hydraulic failures resulting from inadequate delivery of flow and pressure head at demand points decrease WDS reliability (Bao and Mays 1990). A hydraulic failure is defined as a scenario in which a given demand node receives insufficient flow rate under inadequate pressure head. In most studies conducted to examine WDS reliability, the hydraulic reliability index is defined as the probability that the pressure at each node is above the MPC given that adequate demand is supplied (Bao and Mays 1990; Gomes and Karney 2005; Atkinson et al. 2014). Here nodal reliability is defined according to Bao and Mays (1990) as

$$RN = P(H_s \geq \text{MPC} | Q_s = Q_r) \tag{4}$$

where RN = nodal reliability; H_s = supplied pressure head; Q_s = supplied demand; and Q_r = required demand. Because the hydraulic simulator *EPANET2* always satisfies demand but not necessarily pressure head, this approach automatically assumes that water demand is satisfied. Therefore, hydraulic failure is considered to be due to inadequate pressure at demand points. Of course, pressure supplied by WDSs can sometimes be lower than the requirement under deficient service conditions (e.g., pipe outage, power failure at pumping stations, fire flow conditions), and these conditions affect system reliability. Under deficient conditions, pressure-driven analysis (PDA) should be performed to accurately predict system response with pressure deficits (Wu et al. 2009; Jun and Guoping 2013). If, however, nodal reliability [Eq. (4)] is maintained at 100% (i.e., pressures at or above the MPC are supplied for all nodes at all times), demand-driven models such as *EPANET2* can be used to simulate water network performance. Pipe failure also affects system reliability, but the focus of this study is not to determine system reliability; rather, the purpose of the reliability index is to have an indicator for ensuring that minimum pressure at each node is above the MPC as required in the general WDS design. The same method has been used by other researchers (e.g., Babayan et al. 2005; Kapelan et al. 2005, 2006; Bao and Mays 1990). In the last part of the simulation, after all individual MCS runs are performed, the PDF of maximum pressure is determined and the expected pipe break rates are computed based on Eq. (3).

Case Study

The presented probabilistic approach was applied to a part of the City of Hamilton's distribution system in Ontario, Canada. This network consists of 240 nodes, 1 source, and 273 pipes (Fig. 4). Three parallel pumps are connected to the source. The average total system demand is 593 L/s, and 27 nodes have no demand. To model diurnal fluctuations, a diurnal demand pattern, depicted in Fig. 5, was applied to all nodes. Three identical pumps in parallel with the characteristic curve defined by $H = 66.7 - 3.8 \times 10^{-4} Q^2$,

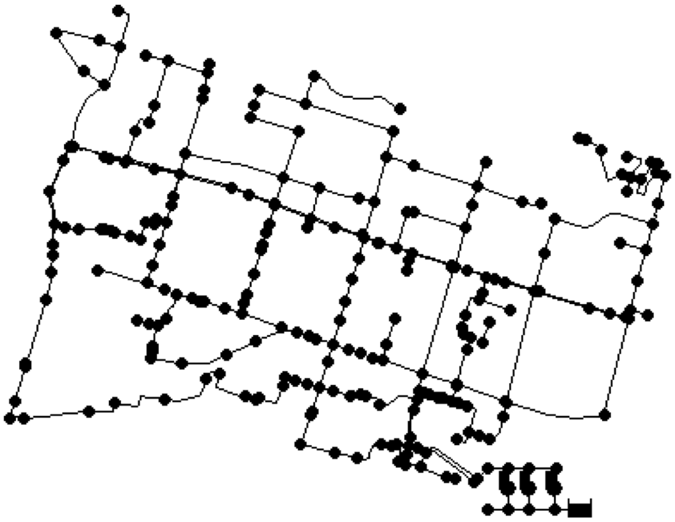

Fig. 4. Northeast portion of the Hamilton network

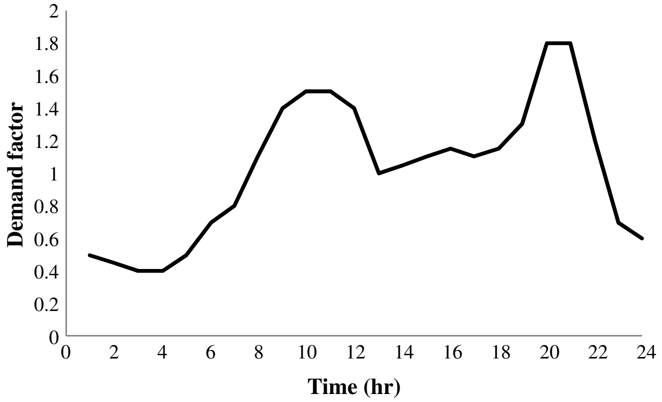

Fig. 5. Diurnal demand pattern for water consumption

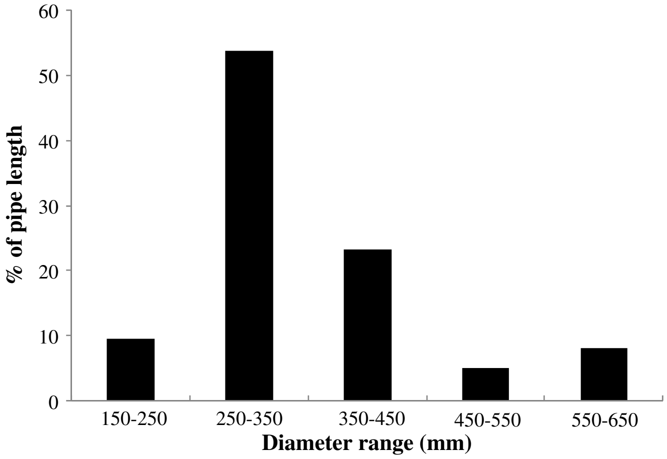

Fig. 6. Percentages of pipe length in different diameter categories

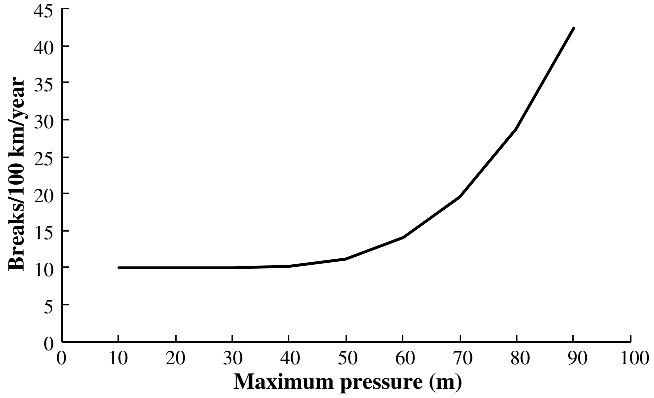

Fig. 7. Pipe break rate function (adapted from Lambert et al. 2013)

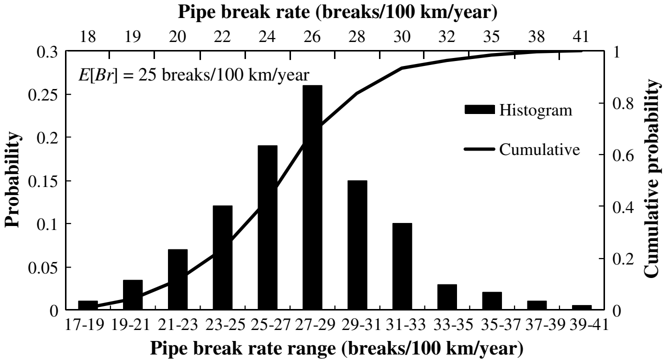

Fig. 8. Histogram and CDF of the Hamilton network's pipe break rates

where H is in meters and Q is in L/s, were considered. On-off pump controls were specified for pump operations. Fig. 6 presents the percentages of pipe length in different diameter categories in the Hamilton network. The vast majority of pipes have diameters less than 450 mm (86.7%). For the case study presented here, all pipes were grouped together and one pipe break rate function was considered. It may be practical to assign different break rate functions to different diameter groups to obtain a proper estimate of expected pipe break rates if sufficient data on pipe breaks are available. Because there were no data on pipe breaks for the Hamilton network, the pipe break rate function (Fig. 7) was adapted from Lambert et al. (2013).

Random number generation was used to generate the uncertain design parameters of demand and pipe roughness. An MCS comprising 1,000 ($N = 1{,}000$ in Table 1) independent runs, each with generated random parameter values, was run for 24 h. To determine the number of runs required in a MCS, samples of different sizes of uncertain parameters were first generated. Then the mean and the standard deviation of each sample were computed. The results revealed that the statistics of samples and simulation results change very slightly when the sample size exceeds 1,000; therefore, 1,000 was selected as the appropriate sample size. The mean of the normal distribution for a demand was set to be the base demand value specified at each node, and the standard deviation was set to be 10% of the mean (therefore, the coefficient of variation $C_v = 0.10$). Pipe roughness was also assumed to follow Gaussian distributions with means equaling the corresponding initial values specified for each pipe and standard deviations equaling 5% of the corresponding mean values. This 5% was selected based on the fact that the generated roughness coefficients fall between 70 and 130, which is the expected range of variation in pipe roughness coefficients during the service life of the pipes. The C_v of 0.10 for demand was selected following the same reasoning. All nodal demands were considered to be independent of each other. Also, nodal demands and pipe roughnesses were assumed to be independent of each other. Because real water systems in developed countries have reliabilities greater than 0.999 (Bao and Mays 1990), pump sizes for this case study were selected to achieve the nodal reliability of 100%.

For the first MCS, the MPC of the Hamilton system was set at 30 m. The simulation results (as shown in Table 1) were used to determine the frequency distribution of the maximum pressure. Multiplied by the break rate function, the maximum pressure frequency distribution was transformed into the pipe break rate frequency distribution. Fig. 8 shows the resulting histogram and cumulative distribution function (CDF) of Hamilton's pipe break rates. The expected rate for this case as determined from the resulting histogram or CDF was 25 breaks/100 km/year. From the CDF curve depicted in the figure, the probability of the break rate being less than 20 breaks/100 km/year was found to be 0.11.

One of the main objectives of WDS design is to ensure that pressures across a network are always between the minimum and maximum acceptable limits. The average maximum and minimum pressure in the Hamilton distribution system over the MCS runs were also calculated. The average maximum and minimum pressures were found to be 76.5 and 43 m, respectively. The value of the average minimum pressure clearly indicated that, although an MPC of 30 m was enforced for all simulations, the average minimum pressure in the system was still much higher than the MPC considered at the design stage. This finding may motivate utilities to extend pressure management to include strategies that decrease operating pressure by reducing pressure standards. This issue has yet to be investigated.

Expected Pipe Break Rates, System Pressure, and MPC

The implicit objective of enforcing an MPC is to provide adequate flow to control fire that may erupt anywhere in the network, to possibly prevent low or negative pressures during transient events and to ensure customer satisfaction with the prevention of low-pressure events. To achieve reasonable operating conditions, different local standards for water pressures are set. For example, the MPC is 14 m in most Canadian provinces whereas it is 20 and 10 m in Australia and the United Kingdom, respectively (Ghorbanian et al. 2015). There are no universally acceptable or established rules or guidelines for specifying MPCs (Ghorbanian et al. 2015). Thus, there may be the possibility of revising the MPC for some WDSs. Reduction in the MPC may cause consumer complaints and make the system more susceptible to low/negative pressures during transient events; however, the beneficial effects of lowering MPC include decreases in water demand, energy use, leakage, and pipe break frequency. Thus, although it may not have been part of the original intent, there is a connection between MPC and pipe break rates. Quantification of this connection would highlight the benefits of MPC reduction. A case study in the United States indicated that approximately one-third of 36 utilities considering pressure management practices maintained average pressures greater than 50 psi at low-pressure locations, suggesting the potential for pressure reduction through pumping at lower heads (LeChevallier et al. 2014). Investigation of the consequences of MPC reduction for consumer satisfaction and WDS hydraulic performance is definitely needed but is beyond the scope of this paper.

The simulation results considering two MPC values for the Hamilton network are summarized in Table 2. As expected, a higher MPC results in higher expected pipe break rates because the expected break rate is assumed to be a monotonically increasing function of pressure. The results in Table 2 also suggest that when the MPC is reduced from 30 to 20 m, average minimum pressures, average maximum pressures, and expected pipe break rates decrease by 24.4, 12.4, and 24%, respectively. Table 2 clearly shows that the probability of the break rate being less than 20 breaks/100 km/year increases from 0.11 to 0.91 when the MPC is reduced by one-third.

Fig. 9 shows the maximum pressure CDFs for the Hamilton distribution system when two MPC values are considered. As expected, reduction in MPC causes a decrease in system pressures during low-flow conditions. The probability of the maximum pressure being less than 70 m is 0.9 when the MPC is 20 m; it is 0.1 (which means that the probability of pressure being higher than 70 m is 0.9) when the MPC is set to be 30 m. This indicates the risk of significantly increasing pressures all across the system that would be created by the use of a high MPC. Another concern in WDS design is how frequently low pressures occur. Although the MPC is enforced in the design stage, low pressures close to the MPC may not occur as frequently as is usually expected. Quantification of the frequency of low pressures is important, but no such studies have been conducted. In Fig. 10, the probability distribution of the minimum pressure occurring within the 24-h simulation period is shown for the Hamilton network. In the simulation, the MPC was set to be 30 m. It was found that the probability of occurrence of a minimum pressure between 30 and 32 m is only 1%. Fig. 10 clearly shows that minimum pressures are much higher than the MPC of 30 m in the Hamilton network most of the time. This confirms that the pressure in a WDS may be considerably higher than what is typically required. However, design standards require that the MPC be met across the network even though pressures as low as the MPC very rarely occur.

Table 2. Expected Pipe Break Rates, Average Minimum and Maximum Pressures, and Probabilities of a Break Rate Less Than 20 breaks/100 km/year for Two Values of MPC

Indicators	MPC = 30 m	MPC = 20 m
Expected pipe break rate (breaks/100 km/year)	25	19
Probability of break rate less than 20 breaks/100 km/year	0.11	0.91
Average maximum pressure (m)	76.5	67
Average minimum pressure (m)	43	32.5

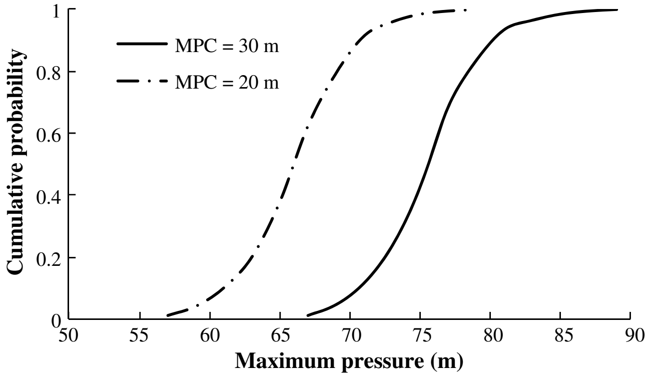

Fig. 9. CDFs of maximum pressures for two values of MPC

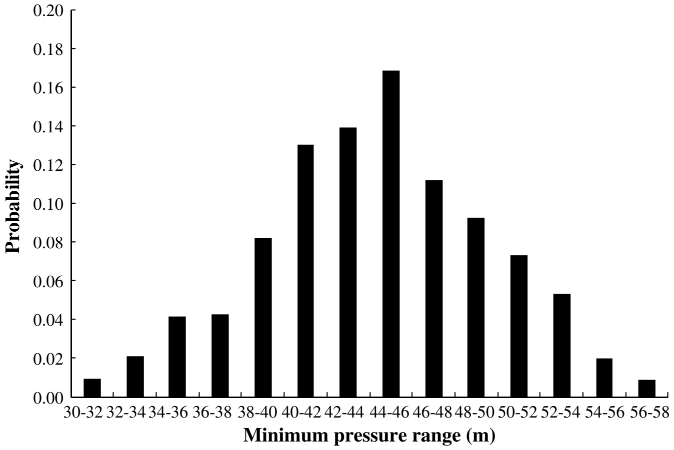

Fig. 10. Probability distribution of the Hamilton network's minimum pressure

Shortcomings and Possibilities of the Proposed Approach

The proposed approach for determining expected pipe break rates associated with system operating pressures currently remains theoretical because of the scarcity of field data. If a relationship between pipe break rates and maximum operating pressures can be established for different WDSs physical and environmental conditions, development of economic models to determine the financial benefit of pressure reduction becomes possible. This gives support to the argument for collecting data in different networks to determine the exact form of the break rate function. Although many large municipalities are now collecting demand and pipe break data with SCADA systems, system pressure also needs to be recorded to determine the pipe break rate function. A more realistic approach would be to estimate the leakage time-instant and then estimate the pressure at the leakage location using a network model. An estimate of the leakage time-instant is useful for diagnosis because it may clarify the causes of leakage (Boracchi et al. 2013). Also, to determine whether a pipe break is due to high pressures, field work is required to determine whether failure is due to internal or external causes. It is heartening that municipalities are setting up geographic information systems (GISs) to integrate a network's data for improved visualization and graphical querying of implicit and explicit knowledge accumulated during WDS maintenance and management. With the extensive capabilities of GIS, it is possible to display breaks occurring in any given event together with the corresponding system pressure, and then eventually determine the break rate function. Indeed, the savings in repair costs associated with pipe breaks justify the time, cost, and resources needed to collect the required data.

Conclusions

High operating pressure increases the frequency of pipe breaks. This paper presented a probabilistic approach to quantifying the expected pipe break rates in WDSs. The probabilistic approach, considering uncertain demands and pipe roughnesses, was applied to compute expected pipe break rates in the Hamilton network. This was estimated to be 25 breaks/100 km/year. When the MPC was reduced by 33%, the average minimum and average maximum pressures respectively decreasd by 24.4 and 12.4%, respectively.

For the case study presented in this paper, the probability of the maximum pressure being less than 70 m increased from 0.1 to 0.9 when the MPC was reduced by 33%. The frequency of low pressures was quantified, and it was shown that low pressures (those close or equal to the MPC) occur very infrequently. These findings may motivate water utilities to rethink about pressure standards. The expected pipe break rates defined in this paper can be used as an indicator in WDS design and can also be easily incorporated into an optimization scheme to minimize expected pipe break rates.

A similar probabilistic approach was used by Filion et al. (2007b) to introduce a stochastic design method for quantifying expected annual damages associated with low- and high-pressure hydraulic failures in WDSs. In this paper, however, the context differed from the one studied by those researchers. Only steady-state pressures were considered here; dynamic pressures caused by hydraulic transients or water hammers were excluded. Of course, uncontrolled transient pressures can significantly affect pipe breaks, and the related surge pressures should eventually be incorporated into the proposed probabilistic approach. To achieve this, several challenging studies need to be performed that include two steps: (1) establishing the relationship between magnitude and occurrence frequency of high transient pressures and magnitude and occurrence frequency of maximum and/or minimum operating pressures during a typical day as calculated in the proposed MCS; and (2) obtaining maximum pressure frequency distributions, including both calculated steady-state maximum pressures and transient pressures determined based on the relationship established in Step (1). Step (1) requires extensive observed transient data and likely extensive network transient modeling studies considering all routine and permitted transient events during a typical period of operation. Currently available data sets are typically insufficient or too short-term for the completion of this step. Use of extensive transient modeling, combined with careful calibration studies, is required for the completion of Step (1); subsequently the estimation of expected pipe break rates may be investigated in future studies. The challenge is that the specifics of system responses and of individual pipe breaks are inevitably complex, and that pipe breaks often arise from a combination of interacting factors.

Acknowledgments

The writers wish to thank the Natural Sciences and Engineering Research Council of Canada for its financial support for this research.

References

Achim, D., Ghotb, F., and McManus, K. (2007). "Prediction of water pipe asset life using neural networks." *J. Infrastruct. Syst.*, 10.1061/(ASCE)1076-0342(2007)13:1(26), 26–30.

Ali, M. (2014). "Knowledge-based optimization model for control valve locations in water distribution networks." *J. Water Resour. Plann. Manage.*, 10.1061/(ASCE)WR.1943-5452.0000438, 04014048.

Andreou, S. A., Marks, D. H., and Clark, R. M. (1987). "A new methodology for modelling break failure patterns in deteriorating water distribution systems: Theory." *Adv. Water Resour.*, 10(1), 2–10.

Atkinson, S., Farmani, R., Memon, F. A., and Butler, D. (2014). "Reliability indicators for water distribution system design: Comparison." *J. Water Resour. Plann. Manage.*, 10.1061/(ASCE)WR.1943-5452.0000304, 160–168.

AWWA (American Water Works Association). (2009). "Water audits and loss control programs." Denver.

Babayan, A. V, Savic, D. A., and Walters, G. A. (2005). "Multiobjective optimization for the least-cost design of water distribution systems under correlated uncertain parameters." *World Water Congress 2005: Impacts of Global Climate Change–Proc., 2005 World Water and Environmental Resources Congress*, ASCE, Reston, VA.

Bao, Y., and Mays, L. W. (1990). "Model for water distribution system reliability." *J. Hydr. Eng.*, 10.1061/(ASCE)0733-9429(1990)116:9(1119), 1119–1137.

Berardi, L., Kapelan, Z., Giustolisi, O., and Savic, D. A. (2008). "Development of pipe deterioration models for water distribution systems using EPR." *J. Hydroinformatics*, 10(2), 113–126.

Boracchi, G., Puig, V., and Roveri, M. (2013). "A hierarchy of change-point methods for estimating the time instant of leakages in water distribution networks." *Proc., Int. Conf. on Artificial Intelligence Applications and Innovations*, Springer, Berlin.

Brander, R. (2001). "Water pipe materials in Calgary, 1970–2000." *AWWA Infrastructure Conf. Proc.*, American Water Works Association, Denver.

Campisano, A., Modica, C., and Vetrano, L. (2012). "Calibration of proportional controllers for the RTC of pressures to reduce leakage in water distribution networks." *J. Water Resour. Plann. Manage.*, 10.1061/(ASCE)WR.1943-5452.0000197, 377–384.

Clark, R. M., Stafford, C. L., and Goodrich, J. A. (1982). "Water distribution systems: A spatial and cost evaluation." *J. Water Resour. Plann. Manage.*, 108(WR3), 243–256.

Creaco, E., Lanfranchi, E., Chiesa, C., Fantozzi, M., Carrettinic, C. A., and Franchini, M. (2016). "Optimisation of leakage and energy in the Abbiategrasso district." *Civ. Eng. Environ. Syst.*, 33(1), 22–34.

Creaco, E., and Pezzinga, G. (2014). "Multiobjective optimization of pipe replacements and control valve installations for leakage attenuation in water distribution networks." *J. Water Resour. Plann. Manage.*, 10.1061/(ASCE)WR.1943-5452.0000458, 04014059.

Fanner, P., Sturm, R., Thornton, J., Liemberger, R., Davis, S. E., and Hoogerwerf, T. (2007). *Leakage management technologies*, AWWA Research Foundation, Denver.

Filion, Y., Adams, B. J., and Karney, B. W. (2007a). "Cross correlation of demands in water distribution network design." *J. Water Resour. Plann. Manage.*, 10.1061/(ASCE)0733-9496(2007)133:2(137), 137–144.

Filion, Y. R., Adams, B. J., and Karney, B. W. (2007b). "Stochastic design of water distribution systems with expected annual damages." *J. Water Resour. Plann. Manage.*, 10.1061/(ASCE)0733-9496(2007)133:3(244), 244–252.

Ghorbanian, V., Karney, B., and Guo, Y. (2015). "Minimum pressure criterion in water distribution systems: Challenges and consequences." *Proc., EWRI 2015: Floods, Droughts, and Ecosystems: Managing Our Resources Despite Growing Demands and Diminishing Funds*, EWRI, Austin, TX.

Gomes, J., and Karney, B. W. (2005). "Water distribution system reliability under a fire flow condition: A probabilistic approach." *World Water Congress: Impacts of Global Climate Change–Proc., 2005 World Water and Environmental Resources Congress*, ASCE, Reston, VA.

Gomes, R., Sa Marques, A., and Sousa, J. (2011). "Estimation of the benefits yielded by pressure management in water distribution systems." *Urban Water J.*, 8(2), 65–77.

Hughes, D. M., Kleiner, Y., Rajani, B., and Sink, J. (2011). *Continuous system acoustic monitoring: From start to repair*, AWWA Research Foundation, Denver.

Iman, R. L., and Conover, W. J. (1982). "A distribution-free approach to inducing rank correlation among input variables." *Commun. Stat. Simul. Comput.*, 11(3), 311–334.

Jacobs, P., and Karney, B. W. (1994). "GIS development with application to cast iron main breakage rate." *Proc., 2nd Int. Conf. on Water Pipeline Systems*, BHR Group, Edinburgh, Scotland.

Jowitt, P. W., and Xu, C. (1990). "Optimal valve control in water distribution networks." *J. Water Resour. Plann. Manage.*, 10.1061/(ASCE)0733-9496(1990)116:4(455), 455–472.

Jun, L., and Guoping, Y. (2013). "Iterative methodology of pressure-dependent demand based on EPANET for pressure-deficient water distribution analysis." *J. Water Resour. Plann. Manage.*, 10.1061/(ASCE)WR.1943-5452.0000227, 34–44.

Kang, D. S., and Lansey, K. (2009). "Real-time demand estimation and confidence limit analysis for water distribution systems." *J. Hydr. Eng.*, 10.1061/(ASCE)HY.1943-7900.0000086, 825–837.

Kang, D. S., Pasha, M. F. K., and Lansey, K. (2009). "Approximate methods for uncertainty analysis of water distribution systems." *Urban Water J.*, 6(3), 233–249.

Kapelan, Z., Savic, D. A, Walters, G. A., Babayan, A. V. (2006). "Risk- and robustness-based solutions to a multiobjective water distribution system rehabilitation problem under uncertainty." *Water Sci. Technol.*, 53(1), 61–75.

Kapelan, Z., Savic, D. A., and Walters, G. A. (2005). "Multiobjective design of water distribution systems under uncertainty." *Water Resour. Res.*, 41(11), 1–15.

Kettler, A. J., and Goulter, I. C. (1985). "An analysis of pipe breakage in urban water distribution networks." *Can. J. Civ. Eng.*, 12(2), 286–293.

Kimutai, E., Betrie, G., Brander, R., Sadiq, R., and Tesfamariam, S. (2015). "Comparison of statistical models for predicting pipe failures: Illustrative example with the City of Calgary water main failure." *J. Pipeline Syst. Eng. Pract.*, 10.1061/(ASCE)PS.1949-1204.0000196, 04015005.

Kleiner, Y., and Rajani, B. (2001). "Comprehensive review of structural deterioration of water mains: Statistical models." *Urban Water*, 3(3), 131–150.

Lambert, A., Fantozzi, M., and Thornton, J. (2013). "Practical approaches to modeling leakage and pressure management in distribution systems–Progress since 2005." *Proc., 12th Int. Conf. on Computing and Control for the Water Industry*, CCWI, Perugia, Italy.

Lambert, A., and Hirner, W. (2000). "Losses from water supply systems: Standard terminology and recommended performance measures." *The blue pages*, IWA, London.

Lansey, K. E., Duan, N., Mays, L. W., and Tung, Y. K. (1989). "Water distribution system design under uncertainties." *J. Water Resour. Plann. Manage.*, 10.1061/(ASCE)0733-9496(1989)115:5(630), 630–645.

LeChevallier, M. W., Yang, J., Xu, M., and Hughes, D. (2014). *Pressure management: Baseline to optimized utility case studies*, Water Research Foundation, Denver.

Le Gauffre, P. E., et al. (2010). "Impacts of climate change on maintenance activities: A case study on water pipe breaks." *Rep. No. 2014.006*, European Commission, France.

Makar, J., and Kleiner, Y. (2000). "Maintaining water pipeline integrity." *Proc., AWWA Infrastructure Conf. and Exhibition*, AWWA, Denver.

Mallick, K. N., Ahmed, I., Tickle, K. S., and Lansey, K. E. (2002). "Determining pipe groupings for water distribution networks." *J. Water Resour. Plann. Manage.*, 10.1061/(ASCE)0733-9496(2002)128:2(130), 130–139.

Martins, A., Leitao, J., and Amado, C. (2013). "Comparative study of three stochastic models for prediction of pipe failures in water supply systems." *J. Infrastruct. Syst.*, 10.1061/(ASCE)IS.1943-555X.0000154, 442–450.

Martınez-Codina, A., Cueto-Felgueroso, L., Castillo, M., and Garrote, L. (2015). "Use of pressure management to reduce the probability of pipe breaks: A Bayesian approach." *J. Water Resour. Plann. Manage.*, 10.1061/(ASCE)WR.1943-5452.0000519, 04015010.

Morris, R. E., Jr. (1967). "Principal causes and remedies of water main breaks." *J. AWWA*, 54(7), 782–798.

O'Day, D. (1982). "Organizing and analyzing leak and break data for making main replacement decisions." *J. AWWA*, 66(3), 911–917.

O'Day, K. (1989). "External corrosion in distribution systems." *J. AWWA*, 81(10), 45–52.

Ormsbee, L. E., and Lingireddy, S. (1997). "Calibrating hydraulic network models." *J. AWWA*, 89(2), 42–50.

Pearson, D., Fantozzi, M., Soares, D., and Waldron, T. (2005). "Searching for N2: How does pressure reduction reduce burst frequency?" *IWA Conf.: Proc., Leakage 2005*, IWA, Halifax, NS.

Pelletier, G., Mailhot, A., and Villeneuve, J.-P. (2003). "Modeling water pipe breaks–Three case studies." *J. Water Resour. Plann. Manage.*, 10.1061/(ASCE)0733-9496(2003)129:2(115), 115–123.

Rajani, B., and Zhan, C. (1996). "On the estimation of frost load." *Can. Geotech. J.*, 33(4), 629–641.

Rossman, L. A. (2000). *EPANET2 users' manual*, U.S. Environmental Protection Agency, Cincinnati.

Shamir, U., and Howard, C. D. D. (1979). "An analytic approach to scheduling pipe replacement." *J. AWWA*, 71(5), 248–258.

Shinstine, D. S., Ahmed, I., and Lansey, K. E. (2002). "Reliability/availability analysis of municipal water distribution networks: Case studies." *J. Water Resour. Plann. Manage.*, 10.1061/(ASCE)0733-9496(2002)128:2(140), 140–151.

Singh, A., and Adachi, S. (2013). "Bathtub curves and pipe prioritization based on failure rates." *Built Environ. Project Asset Manage.*, 3(1), 105–122.

Speight, V. (2008). "Water-distribution systems: The next frontier." *Bridge J. Natl. Acad. Eng.*, 38(3), 31–37.

Thornton, J., and Lambert, A. (2006). "Managing pressures to reduce new breaks." *Water J.*, 24–26.

Tung, Y. K. (1986). "Model for optimal risk-based water distribution network design." *Proc., Water Forum'86: World Water Issues in Evolution*, ASCE, Reston, VA.

USACE (U.S. Army Corps of Engineers). (1981). "Urban water study–Buffalo, New York." *Final Rep.*, Washington, DC.

Walski, T., Bezts, W., Posluszny, E. T., Weir, M., and Whitman, B. E. (2006a). "Modeling leakage reduction through pressure control." *J. Am. Water Works Assoc.*, 98(4), 147–155.

Walski, T. M., Frank, N. D, Voglino, T., Wood, R., and Whitman, B. E. (2006b). "Determining the accuracy of automated calibration of pipe network models." *Proc., 8th Annual Water Distribution Systems Analysis Symp.*, ASCE, Reston, VA.

Walski, T. M., and Pelliccia, A. (1982). "Economic analysis of water main breaks." *J. Am. Water Works Assoc.*, 74(3), 140–147.

Wang, Y., Zayed, T., and Moselhi, O. (2009). "Prediction models for annual break rates of water mains." *J. Perform. Constr. Facil.*, 10.1061/(ASCE)0887-3828(2009)23:1(47), 47–54.

Wu, Z. Y., et al. (2002). "Impact of measurement errors on optimal calibration of water distribution models." *Proc., Int. Conf. on Technology*

Automation and Control of Wastewater and Drinking Water Systems, Gdansk Univ. of Technology, Gdansk, Poland.

Wu, Z. Y., Rong, H. W., Walski, T. M., Shao, Y. Y., Bowdler, D., and Baggett, C. C. (2009). "Extended global-gradient algorithm for pressure-dependent water distribution analysis." *J. Water Resour. Plann. Manage.*, 10.1061/(ASCE)0733-9496(2009)135:1(13), 13–22.

Xu, C., and Goulter, I. (1999). "Reliability-based optimal design of water distribution networks." *J. Water Resour. Plann. Manage.*, 10.1061/(ASCE)0733-9496(1999)125:6(352), 352–362.

Yannopoulos, S., and Spiliotis, M. (2013). "Water distribution system reliability based on minimum cut–set approach and the hydraulic availability." *Water Resour. Manage.*, 27(6), 1821–1836.