Semana temática: Agua y ciudad

Eje temático: Pautas de los gobiernos locales para la sostenibilidad

Título de la ponencia: *Management and renewal of urban water infrastructure*

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Resumen:

Large-diameter transmission water mains and small diameter distribution mains are essential components of urban water supply systems. Large proportions of these pipes have average ages greater than 50 years and their condition is rarely known with any accuracy. As water mains age, they deteriorate. An immediate question that arises from the utility's perspective is which pipes require attention and when. This question is discussed in terms of *failure management* for small diameter mains and *failure prevention* for large diameter mains.

This paper focuses on the *failure management* and prevention strategies for water mains. The most common measure of deterioration of small diameter distribution mains is the frequency of failure, which can be proactively mitigated using cathodic protection to control external corrosion. This paper highlights the importance and the benefits of data collection and how subsequent analyses using models can help plan the renewal of deteriorated water mains. These analyses can assist water planners to identify which pipes to renew, when and how, subject to service levels and budget constraints. These analyses are discussed in the context of a case study.

Palabras clave: Water mains, failure management, cathodic protection.

1. BACKGROUND AND INTRODUCTION

Over the course of human civilizations, different materials such as stone, copper, earthenware, bronze, lead, wood and clay have been used to manufacture pipes to convey water from source to point of use. Significant use of cast iron pipes commenced in early 19th century as urban water distribution networks developed. Until early 1900s most pipes were made of pit cast iron. Thereafter, manufacturing technologies improved (spun cast iron – 1920s, ductile iron – early 1960s) and new materials (steel – 1940s, asbestos cement – 1950s, prestressed concrete – late 1940s, polyvinyl chloride (PVC) – 1970s and polyethylene (PE) – late 1980s) were introduced. Distribution water mains can be as small as 2" (50 mm) and as large as 12" (300 mm) in diameter. Pipes larger than 12" are used primarily as transmission mains. Municipal water main inventories have different proportions of these pipe materials, depending on cost, local material availability as well as on the judgement of water engineers on the suitability of specific pipe material to local conditions.

As water mains age, they deteriorate. Common measures of deterioration are the frequency of water main breaks, adverse effects on water quality failures and loss of hydraulic capacity. Leaky pipes affect both the hydraulic capacity and the water quality in the distribution system. Leaks increase flow demands and provide a pathway for contaminants to intrude into the network when pipes are de-pressurised for repair or maintenance. Frequent breaks thus also contribute to the likelihood of contaminant intrusion. Rates of deterioration can have large variations as they depend on pipe material, installation practice, quality of installation, environmental conditions (soil type, ground water, climate extremes) and operational conditions (pressure, leak detection, cathodic protection).

The planning of renewal of deteriorated water mains requires answers to questions such as: what is the remaining service life of specific pipes; which criteria should be used to replace or rehabilitate a pipe; what is the optimal timing for renewal; what budget allocations should be planned for the short, medium and long term to maintain an acceptable level of service? Essentially planners want to know which pipes to renew, when and how, subject to service levels and budget constraints.

What makes the decision process challenging for water mains is that mechanisms affecting the performance criteria are often not well understood. It is difficult to define and measure performance, let alone decide what level of performance is acceptable; it is difficult to calculate costs involved to achieve a specific level of performance; and the data on performance and condition of these buried assets is not readily available and difficult or costly to obtain. Add to these the spatial and temporal variabilities inherent in even a moderate-size system, and one might begin to understand the difficulty in providing a holistic decision framework.

2. FAILURE RISK IN A WATER DISTRIBUTION NETWORK

A distribution system can fail in more than one way as discussed above. If failure is broadly defined as the inability (momentary or extended) to meet performance criteria, then modes of failure could include any pressure drop below a specified minimum, any unscheduled service disruption, any event of water safety breach, water aesthetic complaints, etc.

In the context of reliability engineering and risk management, the definition of risk depends on the type of asset or system (Henley and Kumamoto, 1981). For buried pipes one can define the risk of any type of failure as the expected magnitude of the consequences of failure(s), i.e.,

$$\textit{Risk of failure} = E(\textit{failure consequence}) = f(\textit{probability of failure, costs of failure}) \quad (1)$$

2.1. Definition of Failure

Discussion on probability of failure in water distribution network must be preceded by the definition(s) of failure. Physical rupture of a water main is fairly easy to define, i.e., "break" or "burst" failure where a repair intervention is required. Hydraulic failure is usually defined as the inability of the network to supply demand at minimum pressure (e.g., Bouchart and Goulter, 1991).

Hydraulic failure can occur due to one or more of the following: (a) demand is greater than that for which the system was designed (leaks, fire flows can be defined as demand as well), (b) a component in the network fails, (c) deterioration of pipes inner surfaces diminishes the network's hydraulic capacity.

The reliability of a water distribution network has received numerous definitions in the literature (e.g., Wagner et al., 1988a, 1988b; Cullinane et al., 1989; Goulter and Bouchart, 1990; Bao and Mays, 1990 and others), and to date there is no single definition that is universally accepted. Invariably, network reliability is a hybrid measure affected by the network topology (redundancy) and its hydraulic capacity.

Water quality failure in the distribution network is by far the most difficult to define and quantify. There are numerous ways (Kleiner, 1998; Sadiq *et al*., 2003) in which water quality failures can occur including: (a) intrusion of contaminants, (b) regrowth of micro organisms, (c) microbial (and/or chemicals) breakthrough, (d) leaching of chemicals or corrosion products, and (e) permeation of organic compounds. The complexity of the mechanisms leading to some of these failures, exacerbated by the spatial and temporal variabilities in the physical state of the pipes as well as the systems' boundary conditions (physical environment, efficacy of treatment, etc.) makes direct physical modelling very challenging. In addition, most water quality events are not detected in real time, which exacerbates the difficulty to model or even validate the exact cause of a water safety failure.

Water aesthetic failures are typically detected following customer complaints, and subsequently addressed by a lengthy process of elimination that may include network modelling and systematic examination of components at, or upstream of, the failure location.

The lack of consensus on what constitutes a failure often leads to poor record keeping practices. Much research needs yet to be done to define all the different types of failure and how these should be documented. Subsequently, studies of failure causes and frequencies can be undertaken in a more rigorous way.

2.2. Consequence (Cost) of Failure

The cost of a water main failure can comprise three components: (a) direct, (b) indirect, and (c) social costs. While direct costs are relatively easy to quantify in monetary terms, indirect costs may require much more effort, and social costs are often the most difficult to describe and assess. Rajani and Kleiner (2002) summarized the details on specific items within each category.

The magnitude of failure consequence is, strictly speaking, a random value because no two failures have the same consequences. Failures in small distribution mains are usually repaired with little effort and typically collateral damage is relatively small. Failures of large transmission mains are relatively rare, and because only few water utilities attempt to assess total failure damage there are currently insufficient data to assign probability distributions to failure costs. The consequences of hydraulic failures are rarely assessed, except when fire liability is concerned. The consequences of water quality failures receive increasing attention because of media exposure, but rigorous assessments are yet to be published. More research is required to gain a better understanding of the true magnitude of indirect and social consequences of all failure types.

2.3. Probability of Failure

The probability of a water main failure due to structural deterioration can be estimated using mechanistic models that compare stresses acting on a pipe to the pipe's residual strength. These models (assuming they are robust and comprehensive) require a lot of data that are either unavailable or very costly to obtain, for even a modest portion of a distribution network, because of spatial variability. Repeated condition assessments, using non-destructive evaluations (NDE) techniques, can assist in the calibration of some of the parameters of these models, and improve their accuracy. Alternatively, a more manageable approach is to develop empirical relationships between the pipe, its exposure to the external and operational environments and its observed failure

frequency. These empirical models typically over-simplify a complex reality in order to achieve "80% of the answer with 20% of the effort". This goal of 80-20 is not always achieved because of over simplification or because of insufficient historical failure data.

The availability of fast and robust water network simulation programs has facilitated the ability to calculate the probabilities of hydraulic failures. However, difficulties still remain with issues such as network calibration, modelling and predicting demand variations, and modelling and predicting the deterioration of roughness coefficients due to tuberculation and corrosion and their spatial and temporal variations.

The probabilities of water quality failures in distribution systems have yet to be addressed in a rigorous manner. Sadiq et al. (2007) have recently proposed a model based on fuzzy cognitive maps to quantify water quality failures in deteriorating water mains distribution network.

2.4. Risk of Failure

Risk mitigation can be achieved by reducing failure probability and/or consequence . As the distribution system ages, its components deteriorate and the probability of failure increases. This is true for structural as well as for hydraulic failures and many types of water quality failures. In some cases, it can be argued that the cost of failure is also likely to increase over time.

Measures to mitigate the cost side of risk are possible but rather limited in scope. Examples include: timely response by a well-trained pipe repair crew (reduce the cost of repair as well as water loss and collateral damage resulting from a main break); good monitoring program (initiate rapid communication actions to the public to minimise the level of population exposure to the low quality or unsafe drinking water).

Mitigating risk on the failure frequency side has a greater potential because theoretically, one can reduce failure frequency to nearly zero albeit at a very high cost. It follows that a rigorous decision process should find a balance between the risk of failure and the cost to mitigate it. Figure 1 illustrates this concept. As long as the pipe continues to age and deteriorate without renewal, its probability of failure (or failure frequency) increases and the risk increases as well (note that here the risk is expressed in discounted expected cost). At the same time, as pipe renewal is delayed, its discounted cost (or present value) declines.

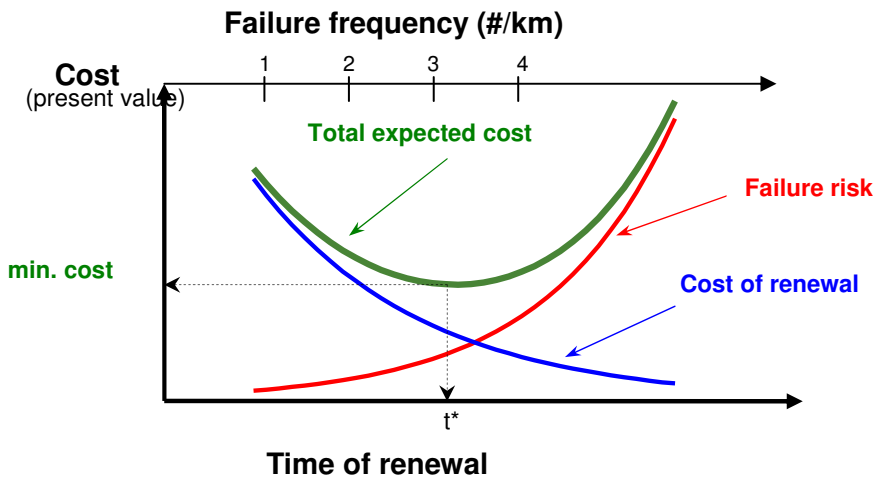

Figure 1. Deciding when to renew a water main with a low cost of failure.

The total expected life-cycle cost is the sum of the total expected cost of failure and cost of renewed pipe. The total expected life-cycle cost curve typically forms a convex shape, whose minimum point depicts the optimal time of renewal (t^*). This point also depicts the time at which the marginal decrease in the discounted cost of renewal equals the marginal increase in the discounted expected risk – this is the balance mentioned above between the risk of failure and the cost to mitigate it. The same type of analysis can be done to include risk mitigation on the failure

consequence side. A similar balance should be sought between the investment required to reduce failure consequence (e.g., build a storage tank in a hospital, or an advanced monitoring system) and the reduction in risk it might achieve.

The top horizontal axis of the graph in Fig. 1 has a notional scale indicating that the optimal renewal time is obtained at a failure frequency of several events per unit length. This represents a typical case of structural failure in small diameter distribution mains, where a given threshold of breakage frequency can be tolerated because the cost of failure is relatively low. This means that the preferable strategy in this case is to pursue *failure management* (frequency of occurrence) rather than attempt to prevent failure altogether.

In Fig. 1 the curve depicting total cost is deeply convex with a clear minimum point at t^*. This is a rather idealised case, which may change in reality. When ageing rate (i.e., the rate at which failure frequency increases) is similar in magnitude to the discounting factor, the convexity of this curve can become quite flat, and the point of minimum cost becomes less crisp. When the cost of failure is relatively low compared to the cost of renewal and the discounting factor relatively high, the curve can take the shape of the "hammock-chair" as described by Herz (1999), with no definite minimum, indicating that renewal should perhaps be postponed indefinitely.

Two points should be highlighted with respect to the convexity of the total cost curve. First, taking into consideration the entire cost of failure, including direct, indirect and social costs, will increase the ratio between the cost of failure and the cost of renewal, which will push the point of minimum towards earlier renewal and increase the convexity of the total cost curve. Second, The discounting factor used should be a social discounting factor, which is invariably lower than a financial one. The social discounting factor can be perceived as a means to distribute available resources over time, or in other words "...discounting acts to distribute benefits today, paid for tomorrow" (Swartzman, 1982). Consequently, the selection of the discount rate reflects the political and ethical attitudes of the decision-maker. The deeper the discounting the more we would tend to reap benefits today and let future generations pay. Selecting a relatively low discount rate will push the point of minimum towards earlier renewal and increase the convexity of the total cost curve.

Figure 2 shows the case of large transmission mains where the ratio between the cost of failure and the cost of renewal is significantly higher. The optimal renewal timing is typically at a failure frequency that is smaller than unity (see notional scale of top axis in Fig. 2). This means that it is economical to take extra measures (and incur extra expense) to try and anticipate failures in order to prevent them before they occur, i.e., *failure prevention*.

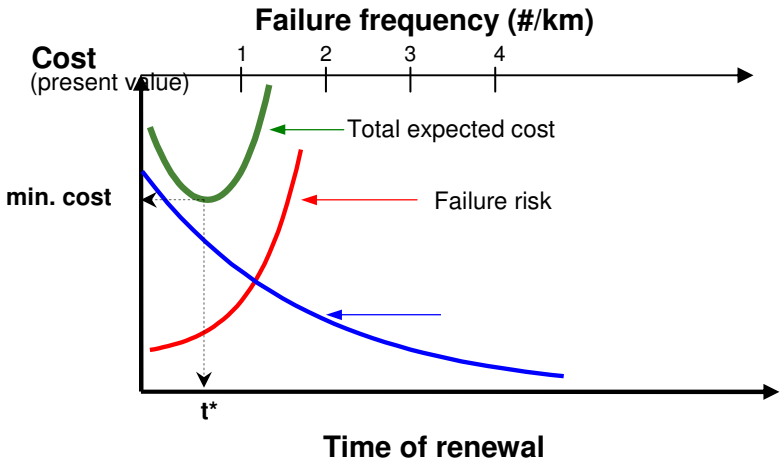

Figure 2. Deciding when to renew a water main with a high cost of failure.

With regard to structural failures, when the cost of failure is relatively low and failure frequency can be tolerated, it is often (but not always) sufficient to rely on empirical models using historical breakage patterns to predict future failure rates. However, high failure costs may justify

the use of extra measures to anticipate failures and prevent them in a proactive approach. These measures could include inspection and condition assessment using NDE techniques in conjunction with physical/mechanical models. Non-destructive evaluations techniques can be used on two levels: first, as a snapshot of the pipe condition at a given time in order to determine if immediate intervention is required, and second, using subsequent inspections to determine the rate of deterioration. It is inevitable that the costs of applying NDE techniques will decrease, as they become widely available. Consequently, their use will become economically viable for larger portions of the distribution system, until eventually all water mains will be periodically inspected by NDE techniques.

3. WATER MAINS: THE IMPORTANCE OF INVENTORY AND FAILURE DATA

Over the past 15 years the National Research Council of Canada (NRC) has been involved in research related to the performance of and decision support for buried infrastructure (water and sewer mains). We have observed that few water utilities are quite proactive and use advanced tools and methods to collect, organise and analyse data, but the vast majority are still lagging. In the current reality of "infrastructure deficit" and diminishing budgets, collection of data for proper planning is a necessary first step in the right direction.

3.1. What data to collect?

Water main data comprise two broad categories, inventory and performance. Inventory data include items such as pipe diameter, material, installation date, length (can be from valve to valve, or one road intersection to another), geographical coordinates, presence and type of service connections, etc. Performance data include failure history, such as date, location (along the pipe) and mode (ring, longitudinal, corrosion hole, on barrel, on bell, etc.) of structural failure; results of hydraulic tests and information about the observed conditions obtained from inspections (visual or from NDE techniques) of pipes.

Collection and organisation of inventory and performance data today is greatly facilitated by the availability of GIS (geographical information systems) and GPS (global positioning systems) and other hand-held devices. Small municipalities that cannot afford to purchase such systems can employ simple procedures for data collection and use spreadsheets to store and organise essential information.

3.2. How to use collected data?

In order to make decisions regarding optimal scheduling of water main renewal, it is necessary to make predictions about their expected deterioration and residual service life. Such predictions are made based on historical performance trends that are gleaned from available data. In other words, an understanding of past performance enables better estimation of future performance. The analyses involved are not too different from those conducted by insurance companies to establish premiums on life and property insurance policies. Recently, software tools have been developed to enable water utilities to perform such data analyses. General understanding can be gained through the study of case histories, however, each water utility must base its predictions on the analysis of its own data to be able to prepare short, medium and long term plans and to make specific renewal decisions. Performance of the same type of pipe can vary significantly depending on local conditions. This, to continue our insurance analogy, is similar to the fact that two identical cars driven by the same person could carry substantially different insurance premiums in two different postal codes.

4. CASE STUDY

In this section we demonstrate the potential use of modeling to plan water main renewal based on the statistical analysis of historical breakage patterns in distribution water mains. In this discussion the term "model" refers to a multi-covariate exponential model proposed by Kleiner and

Rajani (2004), which can consider time dependent (climate, operational) covariates. In the example provided, the time dependent covariates considered are freezing index, rain deficit and cathodic protection.

Cathodic protection (CP) of metallic water mains can often be an effective method to reduce breakage frequency and extend the useful life of pipes. There are two types of CP strategies, hot spot and retrofit. Hot spot cathodic protection (HS CP) is the practice of opportunistic installations of protective anodes at locations of a pipe failure, or other events at which the pipe is exposed. These anodes are typically installed without any means of monitoring and stay in the ground until total depletion, usually without replacement. Retrofit cathodic protection (Retro CP) refers to the practice of systematically protecting existing pipes with galvanic cathodic protection. If an existing water main is electrically discontinuous (e.g., bell and spigot with elastomeric gaskets and no bridging) then an anode is attached to each pipe segment (typically 6 m or 20' length). If the water main is electrically continuous then usually a bank of anodes in a single anode bed can protect a long stretch of pipe. More details can be found in Kleiner and Rajani (2004) and Rajani and Kleiner (2007).

Scarborough (now part of Greater Toronto Area) has been one of earliest water utilities in Canada to recognize the potential usefulness of hot spot and retrofit cathodic. Their hot spot program started between 1983 and 1986 (for the analysis presented here, 1984 was taken as the starting year) while the retrofit program effectively started in 1986, and increased substantially in 1990

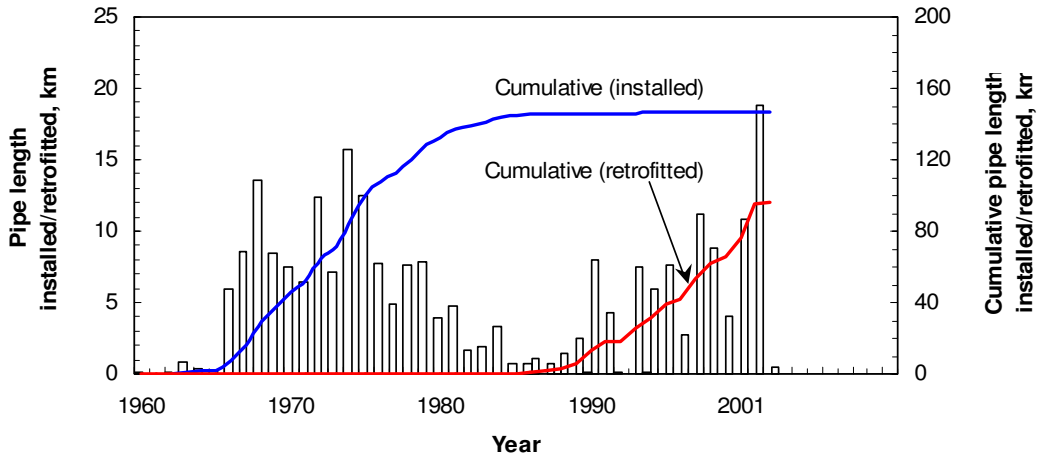

Figure 3. Lengths of 150 mm diameter DI pipes installed (between 1960 and 1994) and retrofitted (between 1994 and 2003) in Scarborough.

Scarborough pipe inventory data included 6,879 individual pipe records (a total of 1,155 km nearly all of which are laid in silty clay soils), each with material type, diameter, installation year and the year retrofit CP was applied. Rajani and Kleiner (2007) presented and analysed a subset (or group) of pipes that consisted of 150 mm ductile iron mains installed between 1960 and 1996 (Fig. 3) and subsequently retrofitted between 1986 and 2003 (with a total length of 146 km). This subset was divided into sub-groups 'A' (50 km) and 'B' (96 km) representing pipes without and with retrofit CP, respectively. Table 1 gives key characteristics for pipes within each of the sub-groups.

Table 1. Principal characteristics of sub-groups (A and B) for breaks on 150 mm diameter ductile iron water mains installed in Scarborough between 1960 and 1996.

Sub-groups	Retrofitted	Pipe length (km)	No of breaks	Average break rate (/100 km/year)
A	No	50	247	11
B	Yes§	96	1,201	28

† Hotpot CP started in 1984; § retrofit CP started in 1986 in Scarborough.

4.1. Break history analysis

The break history analysis for sub-group A is shown in Fig. 4. It consisted of ductile iron (DI) pipes with hot spot CP anodes installed since 1984. The quality of the model "fit" in the break history analysis of sub-group A (adjusted coefficient of determination r^2 = 0.524) is moderately high. It appears that the hot spot CP program has had a dramatic influence (coefficient value of -0.503) in reducing the breakage rate of these ductile iron pipes for the past 20 years. The background ageing rate of 0.133 indicated an annual breakage rate increase (breaks/km) of about 13.3%, which is extremely high compared to the other groups of mains (not discussed here). All climatic covariates except freezing index (*FI*) turned out to have a significant effect on the breakage pattern of sub-group A.

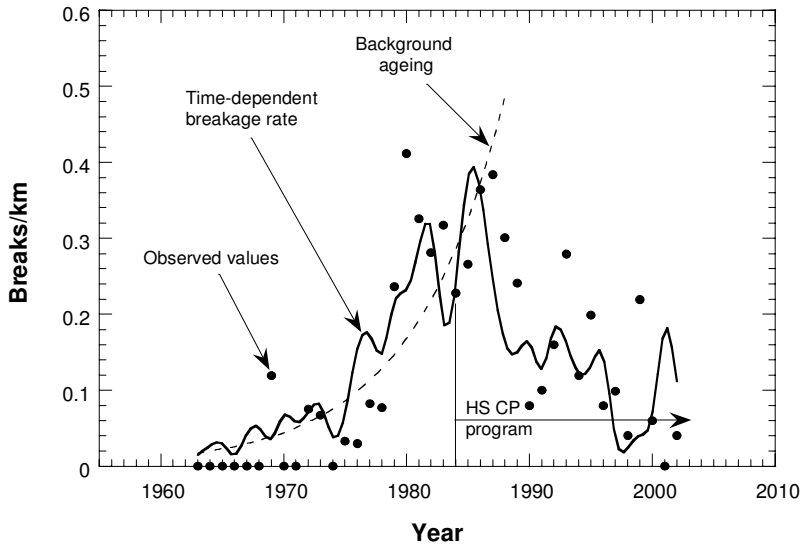

Figure 4. Breakage analysis for sub-group A

The break history analysis for sub-group B is shown in Fig. 5. It consisted of DI pipes that had been gradually retrofitted with anodes since 1986 and had, as well, CP hot spot anodes installed since 1984. The combined effect of retrofit and hot spot strategies for the past 18 years has had a dramatic influence to arrest water main breaks to insignificant levels. The background ageing rate of 0.172 indicated an annual breakage rate increase of about 17.2%, which is about 25% higher that for the ductile iron mains in sub-group A. The overall breakage rate of sub-group B was exceptionally high, which probably explains why this particular group was targeted by the utility for the retrofit cathodic protection. Two concurrent CP programs between 1994 and 2003 protected sub-group B. The hot spot CP had a notable influence in reducing break rates. Nearly 66% of the 96 km of DI pipes in sub-group B were retrofitted between 1995 and 2003 (the last 8 years of the analysis period), which indicates that the hot spot program initiated in 1984 was significantly responsible for the decrease in breakage rates. It is therefore possible that the hot spot retrofit program alone might have been sufficient to affect most of the observed decline in breakage rate of sub-group B. Also the climate covariates had significant contributions to the goodness of fit.

4.2. Planning analysis

Six planning scenarios (Table 2) were generated for sub-group A, with a planning horizon of 30 years. Scenario A1 depicted a 'do nothing' strategy. Scenario A2 depicted a 'hot spot CP only' strategy with no retrofit and no replacements. Scenario A3 depicted the 'minimum' option, in which an average of 4 km of pipe would be retrofitted annually and hot spot CP would continue. Over 30 years of planning horizon, all 50 km of pipes would thus be retrofitted over the first 12.5 years at the rate of 4 km/year. Scenario A4 depicted Scarborough's current (2004) strategy of retrofitting about 12 km of pipe in 2003, but without pipe replacement. Scenario A5 depicted an

'aggressive' strategy, where 20 km of pipe would be retrofitted. Scenario A6 depicted a 'half and half' strategy, where 10 km of pipe would be retrofitted and an equal amount would be replaced annually.

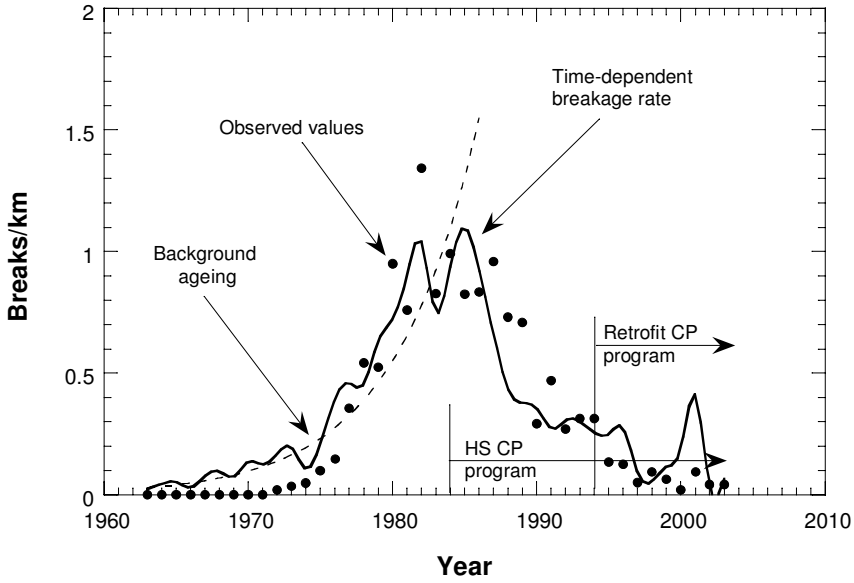

Figure 5. Breakage analysis for sub-group B

Table 2. Planning scenarios for sub-group A (50 km) from Scarborough.

Scenario	Replace-ment	Retrofit	Discounted Costs ($1,000)					Average Breakage rate / 100 km / year
	length / year (km)		Breaks	HS CP anodes	Retrofit CP	Replace-ment	Total	
A1	nil	nil	218,773	0	0	0	218,773	> 3500
A2	nil	nil	3,093	107	0	0	3,200	38
A3	nil	4	1,034	29	2,125	0	3,188	9.3
A4	nil	12	565	11	2,397	0	**2,973**	4.6
A5	nil	20	446	6	2,454	0	**2,906**	3.5
A6	10	10	389	2	998	14,560	15,950	5.1

Table 2 clearly shows scenarios A4 and A5 are superior to the others with low costs and low breakage rates. Scenarios A3 could be considered to be almost as good, given the precision level of this type of analysis. Scenarios A1 and A6 are clearly inferior. Scarborough was in 2004 operating according to scenario A4.

5. FINAL REMARKS

We have argued that from a risk management perspective, it is preferable to pursue a strategy of *managing failures* (frequency of occurrence) of small diameter mains rather than attempt to prevent failure altogether. On the other hand, *failure prevention* is the appropriate strategy for the management of large transmission mains as it is important to try and anticipate imminent failures. Periodic scheduled inspections can help gauge deterioration and thus plan timely intervention before catastrophic (high consequence) failures occur.

The need to collect data on the performance of water distribution network is highlighted. A case study is presented to demonstrate how a model can be used to analyse the collected data to identify trends in the performance of a distribution network. The case history also showed that

cathodic protection programs (hot spot and retrofit) can help reduce breakage rates. These types of models can assist water utilities to better understand the implication of cathodic protection on the performance of distribution networks and optimize the implementation and scheduling of future cathodic protection programs.

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