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Comprehensive Review of Structural Deterioration of Water Mains: Physically Based Models

B. Rajani and Y. Kleiner

Abstract: This paper provides a comprehensive (although not exhaustive) overview of the physical/mechanical models that have been developed to improve the understanding of the structural performance of water mains. Several components have to be considered in modelling this structural behaviour. The residual structural capacity of water mains is affected by material deterioration due to environmental and operational conditions as well as quality of manufacturing and installation. This residual structural capacity is subjected to external and internal loads exerted by the soil pressure, traffic loading, frost loads, operational pressure and third party interference. Some models address only one or a few of the numerous components of the physical process that lead to breakage, while others attempt to take a more comprehensive approach. Initial efforts were aimed mainly towards development of deterministic models, while more recent models use a probabilistic approach to deal with uncertainties in defining the deterioration and failure processes. The physical/mechanical models were classified into two classes: deterministic and probabilistic models. The effect of temperature on pipe breakage is discussed from three angles; the first deals with temperature effects on pipe-soil interaction, the second deals with frost load effects and the third provides a brief review of various attempts to statistically quantify influence of temperature on water main failure.

This paper complements the companion paper "Comprehensive Review of Structural Deterioration of water mains: Statistical Models", which reviews statistical methods that explain, quantify and predict pipe breakage or structural failures of water mains.

Key words: water main failure, physically based models, deterministic, probabilistic, frost load, structural deterioration, temperature effects.

Introduction

The physical mechanisms that lead to pipe breakage are often very complex and not completely understood. These physical mechanisms involve three principal aspects: (a) pipe structural properties, material type, pipe-soil interaction, and quality of installation, (b) internal loads due to operational pressure and external loads due to soil overburden, traffic loads, frost loads and third party interference, and (c) material deterioration due largely to the external and internal chemical, bio-chemical and electro-chemical environment. The structural behaviour of buried pipes is fairly well understood except for issues like frost loads and how material deterioration affects structural behaviour and performance. Consequently, extensive efforts have been applied to model the physical processes of the degradation and failure of buried pipes.

As discussed in the companion paper, more than two thirds of all existing water pipes are metallic (about 48% cast iron and 19% ductile iron). From the 1880s to the early 1930s grey cast iron pipes were manufactured by pouring molten cast iron in upright sand moulds placed in a pit. These pipes were known as pit cast iron. In 1920s/1930s a new manufacturing process was introduced in which pipes were horizontally cast in moulds made of sand or metal that spun as the moulds were cooled externally with water. These pipes were known as spun cast iron, which had better material uniformity than their predecessors, with corresponding improvements in material properties. In 1948 the composition of the iron was changed to produce what is known as ductile iron pipe, which was more ductile and less prone to graphitisation. However, industrial production of ductile iron pipe did not begin until the late 1960s. By 1982 virtually all new iron pipes were ductile iron.

The predominant deterioration mechanism on the exterior of cast and ductile pipes is electro-chemical corrosion with the damage occurring in the form of corrosion pits. The damage to grey cast iron is often disguised by the presence of “graphitisation”. Graphitisation is a term used to describe the network of graphite flakes that remain behind after the iron in the pipe has been leached away by corrosion. Either form of metal loss represents a corrosion pit that will grow with time and eventually lead to a water main break. The physical environment of the pipe has a significant impact on the deterioration rate. Factors that accelerate corrosion of metallic pipes are stray electrical currents, soil characteristics such as moisture content, chemical and microbiological content, electrical resistivity, aeration, redox potential, etc. The interior of a metal pipe may be subject to tuberculation, erosion and crevice corrosion resulting in a reduced effective inside diameter, as well as a breeding ground for bacteria. Severe internal corrosion may also impact pipe structural deterioration. The supply water affects the internal corrosion in pipes through its chemical properties, e.g., pH, dissolved oxygen, free chlorine residual, alkalinity, etc., as well as temperature and microbiological activity.

The long-term deterioration mechanisms in PVC pipes are not as well documented mainly because these mechanisms are typically slower than in metallic pipes and also because PVC pipes have been used commercially only in the last 35 to 40 years. These deterioration mechanisms may however include chemical and mechanical degradation, oxidation and biodegradation of plasticisers and solvents (Dorn et al. 1996).

Asbestos-cement and concrete pipes are subject to deterioration due to various chemical processes that either leach out the cement material or penetrate the concrete to form products that weaken the cement matrix. Presence of inorganic or organic acids, alkalis or sulphates in the soil is directly responsible for concrete corrosion. In reinforced and pre-stressed concrete, low pH values in the soil may lower the pH of the cement mortar to a point where corrosion of the pre-stressing or reinforcing wire will occur, resulting in substantial weakening of the pipe (Dorn et al. 1996).

Pipe breakage is likely to occur when the environmental and operational stresses act upon pipes whose structural integrity has been compromised by corrosion, degradation, inadequate installation or manufacturing defects. Pipe breakage types were classified by O'Day et al. (1986) into three categories: (1) Circumferential breaks, caused by longitudinal stresses; (2) longitudinal breaks, caused by transverse stresses (hoop stress); and (3) split bell, caused by transverse stresses on the pipe joint. This classification may be complemented by an additional breakage type i.e., holes due to corrosion. Circumferential breaks due to longitudinal stress are typically the result of one or more of the following occurrences: (1) thermal contraction (due to low temperature of the water in the pipe and the pipe surroundings) acting on a restrained pipe, (2) bending stress (beam failure) due to soil differential movement (especially clayey soils) or large voids in the bedding near the pipe (resulting from leaks), (3) inadequate trench and bedding practices, and (4) third party interference (e.g., accidental breaks, etc.). The contribution of internal pressure in the pipe to longitudinal stress, although small, may increase the risk of circumferential breaks when occurring simultaneously with one or more of the other sources of stress.

Longitudinal breaks due to transverse stresses are typically the result of one or more of the following factors: (1) hoop stress due to pressure in the pipe, (2) ring stress due to soil cover load, (3) ring stress due to live loads caused by traffic, and (4) increase in ring loads when penetrating frost causes the expansion of frozen moisture in the ground.

This paper reviews physical/mechanical models that lead to an improved understanding of structural performance or behaviour of buried watermains. It complements the companion paper (Kleiner and Rajani 1999), which provides a critical review of the statistical models that attempt to explain, quantify and predict pipe breakage. The two papers share the same format, which provides a description, critique and data requirements for each model. The reader can thus readily identify the principal characteristics of the models with the corresponding limitations and data requirements. Some equations are provided for each models as an illustration of their complexity (or simplicity) and the type of data that is required for their application.

Physical/mechanical Methods

The traditional design of buried pipes has been based on physical behaviour that attempts to provide pipe resistance capacity against expected loads (operational and environmental) with a sufficient margin of safety. The prediction of the mechanical performance of buried pipes with material deterioration requires understanding of several components as is graphically illustrated in Figures 1 and 2. The mechanical behaviour of most of these components is fairly well established and information is available through standards or textbooks (e.g., Young and Trott 1984; Moser 1990) except for recent developments. Therefore, components such as calculation of stresses on pipes subjected to earth and traffic loads are not discussed in detail here. The review is divided into four sections. The first provides a description of some recent work that has been done to model individual components from the array of loadings and degradation mechanisms that act on the pipe. The second and the third sections describe more general models that attempt to capture simultaneously several components. The second and third sections deal with deterministic and probabilistic models, respectively. Efforts to relate pipe breakage rates to temperature effects on a statistical and physical basis are reviewed in the fourth section. The statistically based models are not breakage prediction models in the strict sense since temperatures cannot be predicted in the long-term but do provide an insight into how temperatures have been observed to affect water main breakage rates.

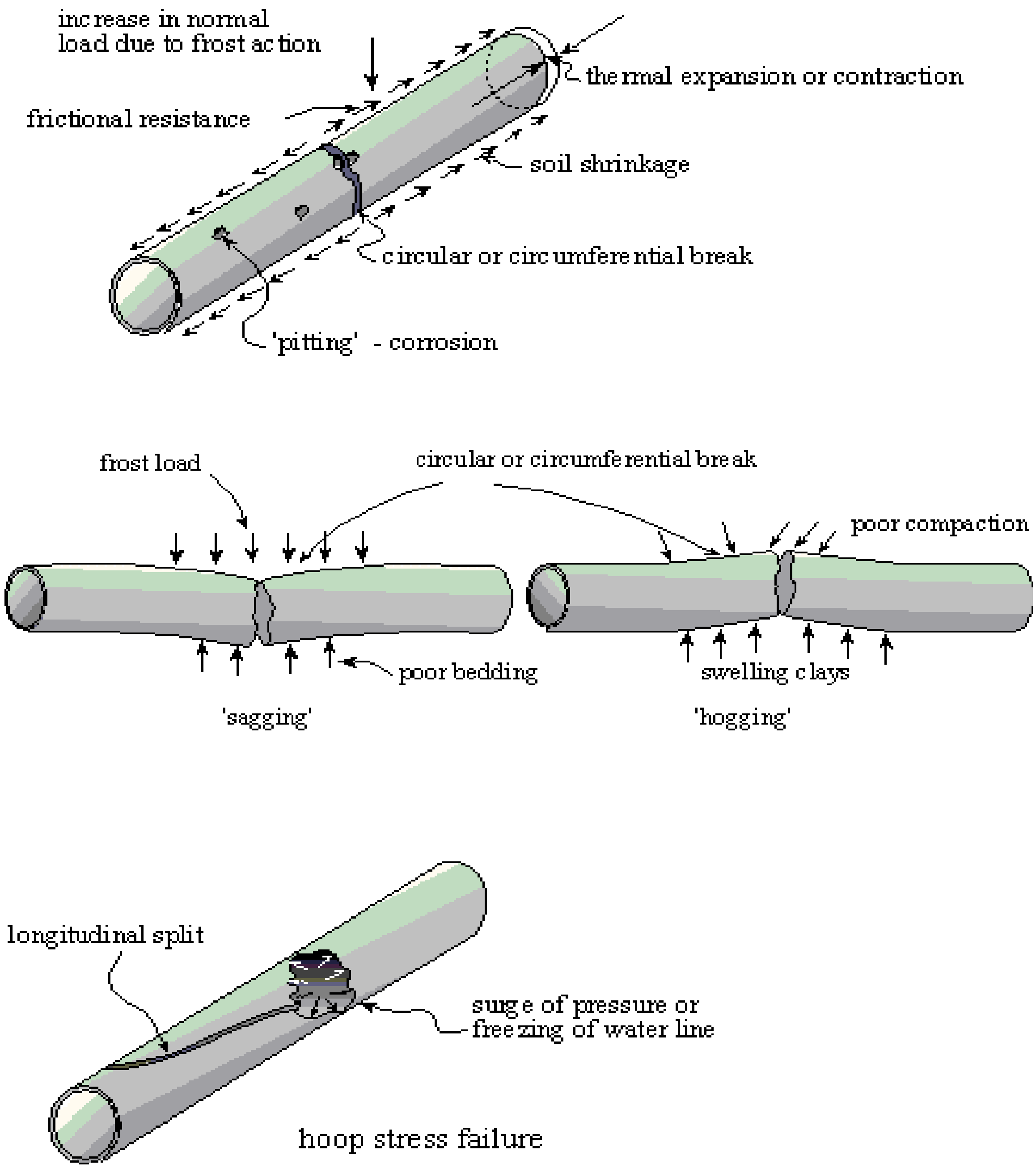

Figure 1. *Failure modes for buried pipes: direction tension (top left), bending or flexural failure (middle) and hoop stress (bottom).*

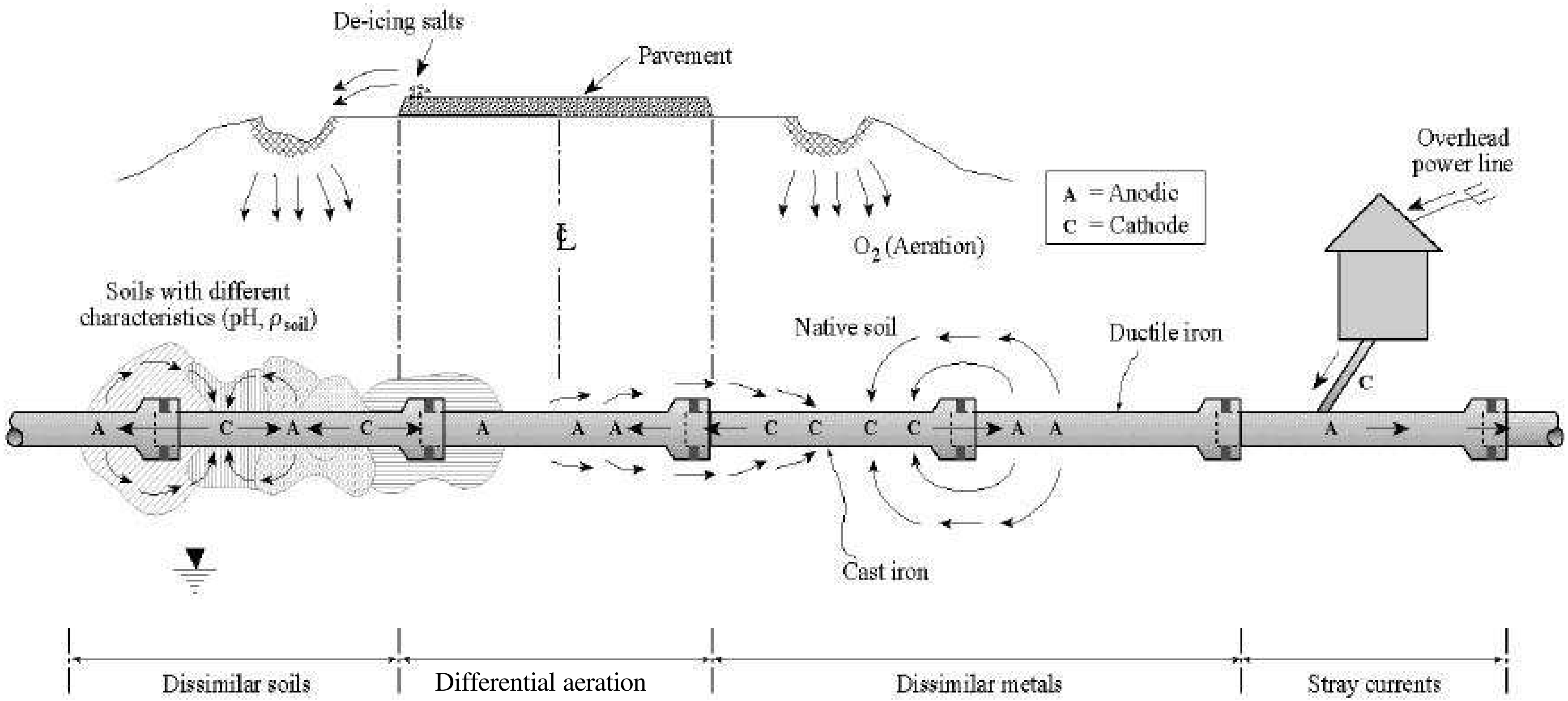

Figure 2. *Conditions leading to corrosion.*

Individual Components of Physically Based Models

Frost load

Model description. The high breakage frequency of water mains during winter has been attributed to increased earth loads exerted on the buried pipes, i.e., frost loads. The mechanics and circumstance that lead to the generation of frost load remained unexplained except by heuristic arguments. Rajani and Zhan (1996) and Zhan and Rajani (1997) presented methods to estimate frost load on buried pipes in trenches and under roadways, respectively. The frost load in a typical trench was calculated from

$$p_s = \sum_{i=0}^{N_T} \frac{\beta h_f^i}{\frac{1}{K_{tip}} + \frac{B_d}{k_s d_f^i}} H\left(s, d_f^i, B_d\right) \tag{1}$$

where p_s = frost load at any point s, d_f = frost depth, i = time step number, N_T = number of time steps, h_f = total frost heave, B_d = trench width, K_{tip} = stiffness of elastic half-space of unfrozen soil mass below freezing front, β = attenuation factor, k_s = backfill-sidewall shear stiffness, H = Boussinesq function to determine the influence of stress, s = location below surface where frost load is calculated.

In a trench, the frost load develops primarily as a consequence of different frost susceptibilities of the backfill and the sidewalls of the trench and the interaction at the trench backfill-sidewall interface. Trench width, differences in frost susceptibilities of backfill and trench sidewall materials, stiffness of the medium below the freezing front and shear stiffness at backfill-sidewall interface play important roles in the generation of frost loads. Thus, it is preferable to use a backfill material that has a matching or lower frost susceptibility than that of the sidewall in order to mitigate against the development of excessive frost loads.

Critique. The frost load models improve the understanding of the mechanisms that lead to the development of frost loads and thus enable the development of mitigating measures. The models are complex and some of the input parameters are not readily available. Although the models have been validated with field measurements, further validations are required to gain confidence in their use. The current field validation data indicate that frost loads could develop to magnitudes of up to twice the geostatic or gravity earth loads.

Data requirements. The models require data such as continuous freezing index, soil backfill properties such as porosity, segregation potential, unfrozen water content and thermal gradient at the freezing front, frost depth and other related variables to predict the frost load on a pipe buried at a particular depth. The segregation potential of backfill is a frost heave property that is conceptually similar to hydraulic conductivity and is not commonly used in civil engineering practice except by specialists in cold regions engineering. The estimates of thermal gradients at the freezing front are dependent on material properties as well as thermal boundary conditions which makes it necessary to use finite element analysis to solve practical trench configurations. The frost load models give frost load as a function of time but only the maximum frost load may be required for most design or structural evaluation procedures.

Pipe-soil interaction analysis

Model description. The interaction of pipe-soil needs to be considered in the *in-plane* and *longitudinal* directions. Spangler (1941) developed an understanding on the interaction of *in-plane* behaviour of flexible pipes with surrounding soils. Watkins and Spangler (1958) improved on the formulation to include the modulus of soil reaction, a property that can be measured. The total circumferential stresses were subsequently obtained by adding the stresses that result from internal pressure, ring compression due to external loads and stress due to ring deflection resulting from static and dynamic loads (e.g., earth pressure, traffic, etc.). The total in-plane stress σ_θ was calculated as

$$\sigma_\theta = \frac{P_i r}{t} + \frac{6 k_m E_p t r}{E_p t^3 + 24 k_d P_i r^3}\left(C_d \gamma B_d^2 + \frac{I_c C_t F}{A} \right) + \sigma_f \tag{2}$$

where C_d, C_t = earth and surface load coefficients, k_m = bending moment coefficient, E_p = pipe elastic modulus, P_i = internal pressure, r and t = mean pipe radius and wall thickness, k_d = deflection coefficient dependent on the distribution of vertical load and reaction, I_c = vehicle impact factor, F = wheel load on surface, σ_f = stress due to pipe ring deflection, B_d = trench width, γ = soil backfill unit weight, A = effective length of pipe.

Rajani et al. (1996) developed a pipe-soil interaction analytical model for the longitudinal response of jointed water mains to changes in internal and external pressures and temperature changes. The response in terms of axial s_x and hoop $s_?$ stresses was expressed as

$$\sigma_x = \chi_1 E_p \frac{\partial u}{\partial x} + \chi_2 P_i - \chi_3 E_p \alpha_p \Delta T \qquad (3)$$

$$\sigma_\theta = \frac{P_i D}{t} h(D/t, E_p/E_s, \nu_p, k_s, \kappa) \qquad (4)$$

where E_p = pipe elastic modulus, u and, x = axial displacement in longitudinal direction x, ν_p = pipe Poisson ratio, α_p = coefficient of pipe thermal linear expansion, P_i = internal pressure, ΔT = temperature differential, E_s = elastic soil modulus, D, t = pipe diameter and wall thickness, k_s = pipe-soil reaction modulus, $\chi_1, \chi_2, \chi_3, \kappa$ = constants as function of soil and pipe properties.

The model for longitudinal response permitted sensitivity analysis to identify key variables that play a major role in the overall behaviour of buried water mains. A sensitivity analysis of ductile iron and PVC water mains indicated that maximum axial stress increased substantially with a decrease in pipe size. This model provided a credible explanation to the observation (e.g., Kettler and Goulter 1985) that under a given set of conditions, small diameter water mains had a higher incidence of breaks per unit length, compared to larger diameter pipes. The model showed that cold ground temperatures can lead to an increase in circular water main breaks and the additional stresses imposed on corroded water mains can be particularly damaging. This model provided a physical explanation to the observations of increased pipe breakage at cold temperatures that were reported by others as discussed later in this paper. The frozen soils or backfill surrounding the buried pipe have a positive restraining effect and thus contribute towards the reduction of hoop stress. Kuraoka et al. (1996) validated the model developed by Rajani et al. (1996) by analysing field data collected on instrumented PVC water mains in Edmonton, Canada.

Critique. The Spangler-Watkins in-plane pipe-soil interaction model assumed that the primary load-resistance action only takes place in the vertical direction, in the plane that is perpendicular to the longitudinal direction of the pipe. This in-plane consideration for the load-resistance action is appropriate for moderate to large diameter pipes and inadequate for small diameter pipes. Moreover, the influence of ground and water temperature was not considered in the Spangler-Watkins model. The longitudinal model developed by Rajani et al (1996) accounted for temperature differentials and explained the frequently observed occurrences of circumferential breaks in small diameter mains. However, none of these models considered the stresses imposed on the mains during extreme dry seasons as a consequence of soil shrinkage.

These pipe-soil interaction models are useful to explain observed behaviour of pipe segments without any degradation such as corrosion pits, thus they cannot, in themselves, be used to predict how the number of water main breaks will vary with time.

Data requirements. The data required by these models are readily available except for the elastic soil reaction modulus and the pipe-soil reaction modulus. The seasonal ground and water temperatures are necessary to determine the influence they exert on the behaviour of the pipe under expected temperature variations. It is also possible to use ambient air temperatures as surrogate variables wherever precise water and ground temperature data are unavailable.

Residual structural resistance

Model description. The assessment of corrosion pits on the structural resistance of thin-walled steel pipes is typically done through the application of ASME/ANSI B31G (1991). This approach was developed semi-empirically, based on experiments where corroded steel pipes were tested to failure. Later, Kiefner and Vieth (1989) conducted additional tests and developed an analytical failure model to predict the pressure at which a pipe with a corrosion pit would fail

$$p_0(d,l) = \frac{hs_y}{r}\left(\frac{1 - A/A_o}{1 - A/(MA_o)}\right) \tag{5}$$

where p_0 = pressure in the pipe at failure , d = maximum depth of corrosion defect, A = cross-sectional area of metal lost in the corroded region projected onto the longitudinal axis of the pipe, A_o = the original cross sectional area of the corroded region, l = maximum total length of corrosion defect, r = pipe radius, h = pipe wall thickness, s_y = pipe yield strength, M = Folias factor that accounts for the bulging of the pipe before failure when subjected to internal pressure.

Critique. This model to assess the reduction in structural resistance in the presence of corrosion pits was developed for ductile materials such as steel pipes that are used primarily in the oil and gas industry. The bulging in the pipe prior to failure (accounted for by the Folias factor) is a phenomenon that is typical to ductile materials only. Since the model has not been validated with pipe materials other than steel, it is not clear whether it is appropriate for cast iron or even ductile iron pipes. The model was developed to represent the bursting failure mode, which in turn corresponds to the longitudinal breaks.

The model requires three-dimensional characteristics of corrosion pits in the pipe. This information for oil and gas pipelines is readily obtainable by using tools known as pigs. The use of these pigs in oil and gas pipelines is increasing because the high cost of failure make their application in frequent inspections economically viable. In the water supply industry similar pigs have recently been developed but they are still in their early stage and are relatively expensive to use. Consequently, it seems that frequent inspections with pigs may currently be warranted only for large transmission mains. It is also possible that as the usage of these pigs increases, the cost will decrease, enabling more frequent inspection cycles.

Data requirements. The data required includes the material properties and condition assessment of a pipe to determine the three-dimensional characteristics of its corrosion pits. Considerable progress has recently been made on practical non-destructive condition assessment techniques to inspect pipe which should facilitate the use of these types of models to assess the longevity of the inspected pipe.

Model description. Rajani et al. (1999) have recently completed an experimental study where coupons of pit and spun cast iron pipe samples with and without corrosion pits were tested. Results from mechanical tests were used to establish how dimensions and geometry of corrosion pits influence the residual strength of gray cast iron mains. The test coupons had corrosion pits with "small" areas in relation to the coupon size. The authors suggested that the nominal tensile stress (σ_n) at which fracture took place depended on the material and corrosion pit dimensions as given by

$$\sigma_n = \frac{\alpha K_q}{\beta\left[\left(d/t\right)\sqrt{a_n}\right]^s} \tag{6}$$

where β = geometric factor dependent on the dimensions and shape of the corrosion pit, a_n = lateral dimension of corrosion pit size, K_q = provisional fracture toughness, d = pit depth, α, s = constants obtained from experimental tests, t = pipe wall thickness. The above expression is essentially the same as the basic fracture mechanics equation that relates stress, defect size and geometry through the stress intensity or toughness of the material.

Critique. The model is based on small-scale laboratory tests and it needs to be validated with large-scale tests. The model also makes use of fracture toughness, a material property that is not readily available for pipe materials of interest. The experimental work was carried out only on brittle material such as cast iron, which is distinct from the work reported by Keifner and Veith (1989) on ductile steels.

Data requirements. The data input requirements are similar to those required in the model suggested by Keifner and Veith (1989).

Corrosion status index

Model description. Kumar et al. (1984) proposed a corrosion status index (CSI) to characterise the condition of cast and ductile iron mains. Although CSI was originally developed for gas mains it also found support (with some modifications) in the water distribution field. The CSI is given by

$$CSI = 100 - 100\frac{P_{av}}{t} \tag{7}$$

where P_{av} = average corrosion pit depth of a 1 m section of pipe, t = pipe wall thickness.

While the CSI of a new pipe is 100, a pipe with average corrosion pit depth of 70% of the original wall thickness will have a CSI of 30. The authors report an empirical observation that the ratio between the average and the maximum corrosion pit depth in a pipe equals 0.7. Further, they reported that the average corrosion pit depth is almost independent of the length or the location of the pipe segment that is examined. Thus, full penetration of the pipe wall would occur for a CSI of 30. The authors then reported formulating “lookup tables” for predicting CSI while considering pipe coating, soil pH and resistivity, age of leak, rate of pit growth, presence of sulphides in the soil and effects of moisture. Although they provided only scant information, it appears that they used a simple power model of the form $D = at^n$ (D is the pit depth, a is a proportion constant, t = time) to predict the basic corrosion rate, where a was modified by other constants representing the effects of the various factors mentioned above. Once a CSI of 30 was reached, a first leak was assumed to have occurred. Kumar et al. (1984) developed a procedure to predict the number of breaks over time, given that the first break has occurred. Their model was based on an exponential increase of breaks over time but they provided very little information as to how they had considered all the factors in this prediction model. A software package “Piper” was subsequently developed for planning and management of pipelines based on these models.

Critique. This model could be also classified as a statistical time-exponential model, as it involves corrosion modelling for predicting the time to the first break, and what appears to be a statistical model to predict subsequent breaks. Kumar et al. (1984) provided few details about their model but the following observations can be made based on the limited available information. Their power model to predict corrosion rate appears to be similar to Rossum's (1969), although no reference was provided. In general the power model requires further validation as discussed in more detail later. Further, the value for the exponent n used is 0.58, which seems high compared to values (0.17 to 0.5) used in the literature (e.g., Rossum 1969). The exponential increase of breaks over time given that a first break has occurred is similar to the model proposed by Clark et al. (1982). A few examples provided by the authors in the report indicate that the breakage rate (subsequent to the first break) increases extremely rapidly. No information is provided on any validation or even "goodness of fit" that would be associated with their predicted versus observed values.

Data requirements. It appears that the data that are required include pipe age, type - wall thickness, diameter, joints soil properties – resistivity, chlorides, sulphides, pH, moisture, and year of first leak (if available).

Physical Deterministic Models

Model description. Doleac (1979) and Doleac et al. (1980) used the power function proposed by Rossum (1968) to relate corrosion pit depth with the pipe age to predict the remaining wall thickness of pit cast mains.

$$p = K_n K_a (10 - pH)^n \rho^{-n} t^n A^a \tag{8}$$

where p = average pit depth, a, K_n, K_a = empirical constants derived from field or lab tests, A^a = pipe surface area exposed to corrosion, pH = soil pH, ρ = soil resistivity, n = soil aeration constant, t = time (years).

The parameter constants were also taken from Rossum (1968). The authors extracted five pipe samples, and their surrounding soil properties in Vancouver, Canada and measured their maximum and average pit depths. Comparison of the maximum and mean pit depth observed to the corresponding values calculated[1] from equation (8) yielded mixed results. Then they substituted the remaining average wall thickness into Barlow's equation of pipe hoop stress

$$S = \frac{pD}{2t} \tag{9}$$

[1] The pipe surface area was not included in these calculations.

where p = internal pressure, D = outside pipe diameter, S = hoop stress on the pipe, t = pipe wall thickness. Pipe failure was defined as "a reduction in pipe wall thickness to a point where a pressure surge in the pipe, equal to 50% of the working pressure, would raise S to the material's elastic limit".

Critique. The model proposed by Doleac (1979) and Doleac et al. (1980) has some shortcomings. There have not been many documented studies that validate Rossum's (1968) power function to predict the growth of corrosion pit depths with time. Doleac's five-sample test is not sufficient to provide significant validation, especially in light of their seemingly mixed results. An attempt to re-create some of the results presented by Doleac (1979) for this review was difficult due to lack of information on values used for some of the parameters in the model. Further, the parameter A (exposed surface area) in equation (8) was used by Rossum (1968) to translate maximum pit depth into average pit depth in a pipe. It is not clear what exposed area should be considered for a given length of pipe, and how an uncoated pipe should be considered. This difficulty may have been the reason why Doleac did not consider parameter A at all in the model. Another limitation is that the structural pipe capacity is based solely on pressure requirements and does not adhere to all the requirements of the standards (AWWA/ANSI C101-67 (R1977)) in existence when these mains were first installed, e.g., external loads, etc.

Data requirements. The data that are required for this model are pipe age and surrounding soil properties, such as resistivity, pH and aeration constant, which can be obtained relatively easily and economically. The exposed surface area of the pipe needs clarification and how it should be calculated or estimated in practice.

Model description. Randall-Smith et al. (1992) proposed a linear model based on an assumption that corrosion pit depth has a constant growth rate (often referred to as corrosion rate), to estimate remaining service or residual life of water mains.

$$\rho = \left(\frac{t}{P_e + P_i}\delta\right) - t \qquad (10)$$

where ρ = remaining life, t = age of water main, δ = thickness of original pipe wall, P_e = external pit depth, P_i = internal pit depth.

The model was developed as a rough screening tool to identify potential problems rather than provide a means to predict a break. It was argued that since all the calculations were based on the most pessimistic assumptions, a result indicating remaining life that exceeded the planning horizon meant that failures from corrosion pitting were unlikely to occur within the planning period. Conversely, if the calculated remaining life was shorter than the planning period, this would indicate that it might be necessary to do rigorous analysis. It was recommended that mean or median corrosion pit depths should be used in the equation (10) for grey cast iron mains, while maximum pit depths should be used for ductile iron mains. This recommendation followed from the recognition that grey iron mains fail as a consequence of the brittle nature of the materials and ductile iron mains fail by developing “pin holes”.

Critique. The assumption of a constant corrosion rate over the life of the pipe is questionable. The assumption about the external and internal corrosion pits coincide at the same location is highly unlikely because the conditions and mechanisms that promote internal and external corrosion are different and independent. This assumption leads to very conservative estimates for the remaining service life of water mains, which indeed was the intention of the authors. However, the model assumes failure when the sum of the internal and external corrosion pit depth equals wall thickness. This assumption considers neither stresses acting on the pipe nor its stress resisting capacity. Since the structural integrity of the pipe can be breached without full perforation of its wall, the model may not be as conservative as it may seem, thus it may not even be suitable for a crude screening. Further, the model expresses the remaining service life as a multiple of the current pipe age, which is neither intuitive nor useful.

Data requirements. The model requires the pipe age, wall thickness and depths of internal and external corrosion pits obtained during the condition assessment of the pipe. The exhumation of many pipe samples may not be economical to measure corrosion pit depths. As previously noted, progress on practical non-destructive condition assessment techniques may soon encourage measurements of corrosion pit depth on a routine basis.

Model description. Rajani and Makar (1999) described a methodology to estimate the remaining service life of grey cast iron mains by considering changes in the structural resistance of a pipe as a result of corrosion pits. They defined the “time of death” of an individual pipe segment as the time at which its mechanical factor of safety fell below a minimum acceptable value set by the utility owner. They calculated the residual resistance of grey cast iron mains based on corrosion pit measurements while explicitly considering anticipated corrosion rates. The calculations combined elements from equations (1), (3), (4), (6), (8) and (9). Direct inspection or non-destructive evaluation technology could be used to measure corrosion pits. The methodology comprised all the major components identified in Figure 3 that should ideally be included in a physical deterministic model. The methodology used a re-iterative procedure to estimate the remaining service life of segments of grey cast iron pipe. The block diagram in Figure 4 illustrates the procedure.

The proposed methodology determined base conditions using pipe information (diameter, wall thickness, date of installation, depth of burial, pipe type – spun or cast); soil condition (type, pH, density, resistivity, aeration quality); installation information (laying condition, load factor, coefficient of horizontal stress at rest, coefficient of sliding friction); operational conditions (water pressure, surge pressures, summer and winter air and water temperatures, wheel loads, vehicle impact factor, frost load factor).

The water utility needs to choose a value for the minimum factor of safety for each pipe segment in its system as part of the initial decision making process. A factor of safety of one signified imminent failure, whereas a factor of safety greater than one, say 1.2, might provide leeway to repair or replace pipe segments before failure occurred.

The methodology proposed two feasible approaches to determine corrosion pit characteristics of grey cast iron mains, an indirect approach using measurements taken by non-destructive evaluation (NDE) technology, or a direct approach where corrosion pit measurements are performed on exhumed pipe samples.

The typical lack of historical corrosion rate data prompted the authors to propose two options to estimate the remaining service life of grey cast iron water mains. The "one-time pit measurement" option provided an estimate based on the initial pipe condition and one more corrosion pit measurement. These two points would then be used in a Rossum-like (1968) power model to approximate the prevalent corrosion rate. The "multiple-time pit measurement" option can provide a more refined estimate of the remaining service life by basing the corrosion rate estimates on a more extensive set of data. The choice between the options would depend on when the methodology is first applied and on the number of pit measurements available. The dimensions of each significant corrosion pit or its expected growth rate is combined to calculate the residual tensile strength. The time required for the factor of safety of the pipe segment to fall below the target value set by the utility is calculated iteratively.

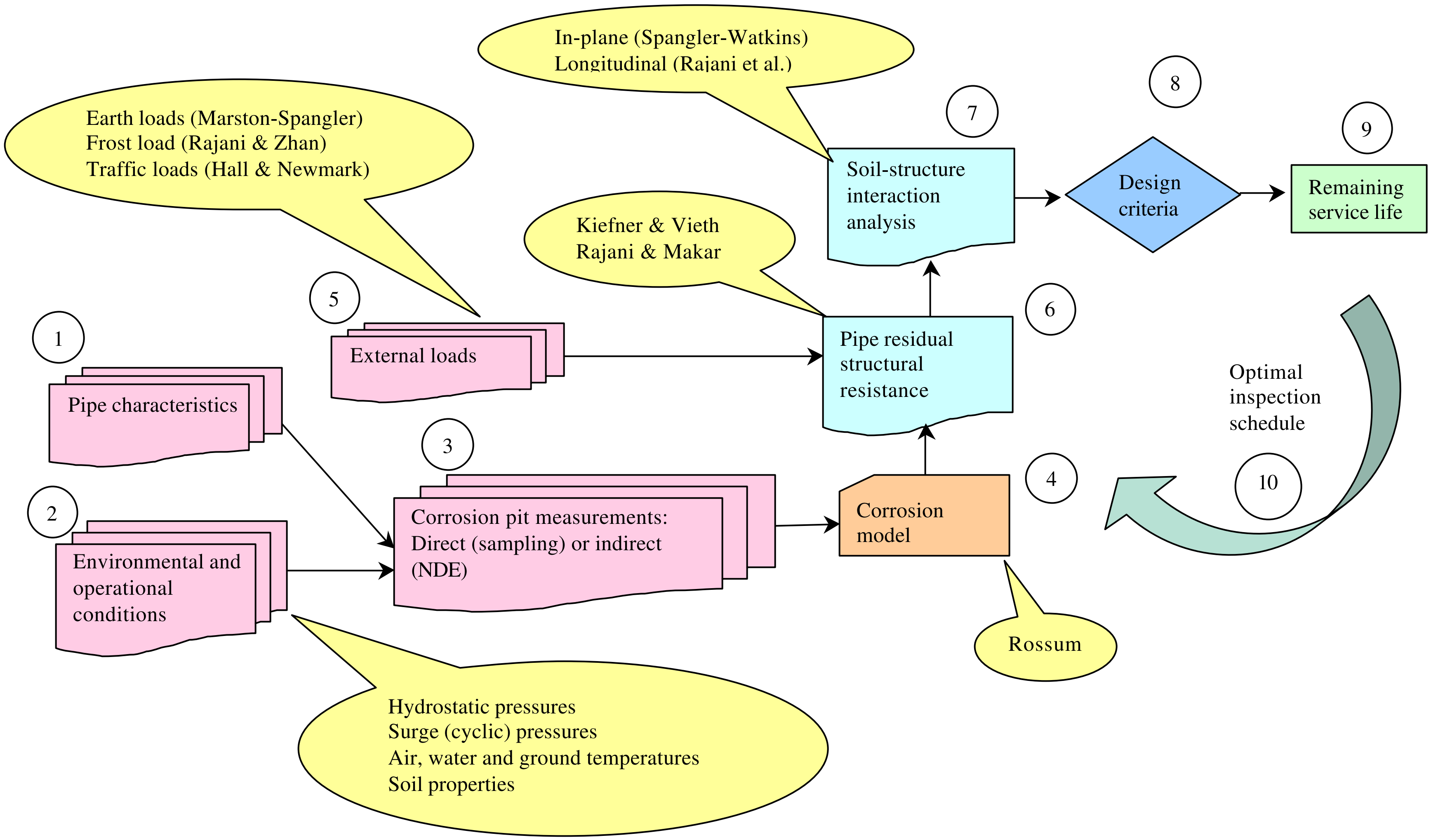

Figure 3. *Physical/mechanical models.*

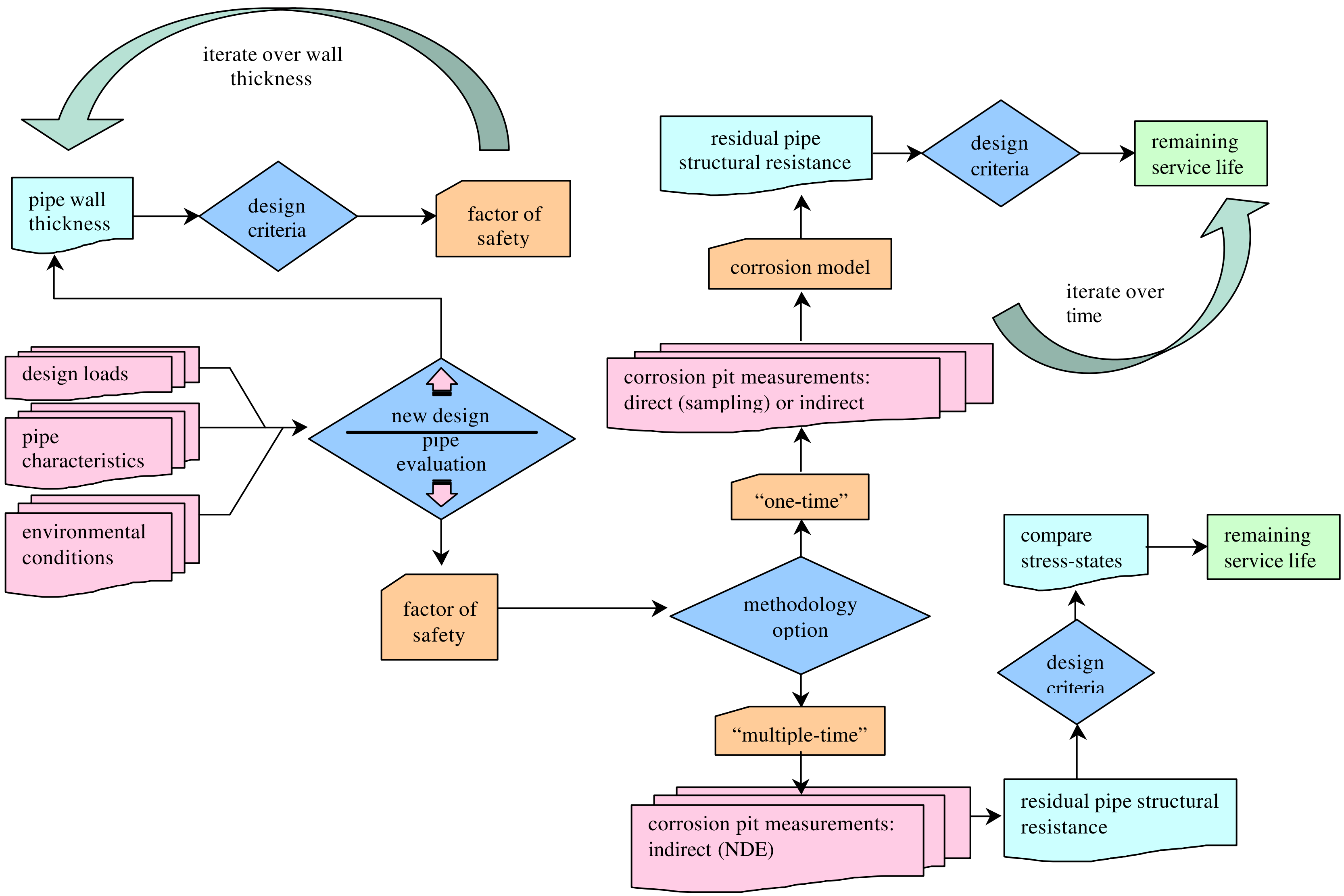

Figure 4. *Physical deterministic model proposed by Rajani and Makar (2000).*

Critique. The methodology proposed by Rajani and Makar (1999) relies on the measurement of corrosion pits by direct inspection or indirectly through the use of non-destructive techniques. The opportunity to directly measure corrosion pits may not always be available because of economic or operational conditions, whereas NDE techniques are currently not developed enough to provide accurate geometry of corrosion pits. The methodology is similar in concept to the physical probabilistic models developed for oil and gas pipelines discussed later except that all the input and output are in deterministic terms. Rajani and Makar (1999) point out the uncertainties in estimating corrosion rates, and the significant impact of these uncertainties on the prediction of the remaining service life of water mains. This conclusion is in line with the conclusion reached by Ahammed and Melchers (1994) in the development of the physical probabilistic model described later. The methodology needs to be validated and possible improvements could incorporate some probabilistic formulations to explicitly account for the uncertainties inherent in the data.

Data requirements. A variety of background data are necessary to define the base condition. The model requires the material properties, current pipe age, wall thickness and internal and external corrosion pits depths to estimate the remaining service life of each pipe or pipe section. The specific material properties required are tensile strength, fracture toughness, elastic modulus and flexural strength. The exhumation of many pipe samples may not be an economical option to measure corrosion pits depths in small distribution pipes. Progress on practical non-destructive evaluation techniques should encourage depth measurements of corrosion pits on a routine basis.

Physical Probabilistic Models

Model description. Ahammed and Melchers (1994) described a model to estimate the probability of failure in steel pipelines. In their model they used equation (2) of the Spangler-Watkins in-plane pipe-soil interaction model as their underlying mechanical stress model[2]. For the wall thickness variable in equation (2) they used a simple power function, $D = kt^n$, where D is the loss of wall thickness at time t, and k and n are regression parameters. Thus they obtained an equation that related the in-plane tensile stress to the age of the pipe. Each parameter and independent variable in the model was then assumed to have a probability distribution with a

[2] The stress due to pipe ring deflection component was not considered by Ahammed and Melchers (1994).

known mean and variance. A second-moment description method was then used to approximate the mean and variance of the dependent variable, namely the in-plane tensile stress. Subsequently, the reliability index of the pipe was derived, and the contribution of each parameter to the uncertainty (or the variance) of the total reliability could be evaluated. A sensitivity analysis indicated that remaining service life was influenced primarily by the parameters involved in the power function for corrosion estimates. This influence was on both the magnitude (mean) of the reliability and on the uncertainty of the reliability. In a subsequent publication, Ahammed and Melchers (1995) extended this model to include leakage of fluids through corrosion pits. Leakage in pipelines was modelled as an exponential function of time and corrosion pitting rate. The failure associated with leakage was that which exceeded a pre-defined limit.

Critique. The models developed by Ahammed and Melchers (1994, 1995) fall within the probabilistic framework of previously developed mechanical models. This probabilistic approach provides insight into the contribution of each parameter to the uncertainty of the results, which is ignored in the deterministic models. There may be some concern with the fact that the second-moment description method, when used with a first-order approximation, is suitable mainly for reliability functions that are relatively linear about the point of failure. This may not always be the case as it may depend on the value of the exponent of the power function. The model in itself suffers from the limitations listed previously for the Spangler-Watkins in-plane pipe-soil interaction model, and the power function as a model for corrosion rate. These limitations include the exclusion of pipes made of brittle material, consideration of stresses only in the circumferential direction (in-plane analysis) and not longitudinal, and the difficulty in deriving the parameters for the corrosion power function. Further, this model does not consider the stress due to pipe ring deflection, which may sometimes be significant. The proposed leakage failure criterion may be practical for the oil and gas industry, where flows are tightly monitored and controlled. In the water industry this criterion seems unrealistic especially in North America, where rigorous water metering is currently practised by few water utilities.

Data requirements. The data required are similar to those required for the Spangler-Watkins in-plane pipe-soil interaction model, plus the parameters for the corrosion power function. The mean values as well as variances (or standard deviations) are required for all data. Most of these properties are readily available except for corrosion parameters in the power function described earlier. Since the model is most sensitive to corrosion parameters, it is paramount to obtain reliable corrosion parameters and that may not always be easy to do in practice. If the actual or approximate probability density functions of these parameters were known, it would be possible to perform Monte-Carlo simulation and obtain results that are more accurate. The Monte-Carlo simulation would also give an idea of how much precision is sacrificed in using the second moment approximation method.

Model description. Several probabilistic physically based models, e.g., Hong (1997, 1998), Jones (1997a, 1997b), Linkens et al. (1998), Pandey (1998), Stephens (1994a, 1994b), have been proposed, that use the residual strength of pipelines suggested by Keifner and Vieth (1989). Two of these models developed for the condition assessment of oil and gas pipelines are discussed briefly here.

Pandey (1997) presented a general probabilistic framework to estimate reliability by incorporating the impact of inspection and repair activities planned over the service life of a pipeline vulnerable to corrosion. The intent of this model was to schedule the optimal inspection interval and repair strategy while maintaining adequate reliability throughout the service life of the pipeline.

Hong (1997) suggested that the ratio between the true (observed) remaining pipe strength and the predicted value $p_0(d,l)$ (equation 5) is a random variable C_m that is approximately log-normally distributed. Hong (1998) developed a probabilistic expression for the load resistance ratio, which is the ratio between the operating pressure acting on the pipe and the pipe remaining strength under pressure. This probabilistic expression comprised the probability distributions of C_m as well as the corrosion pit dimensions d and l. He then proposed a Markov process in which initial state of the pipe was determined following a non-destructive inspection procedure. The deterioration of the load-resistance factor was modelled as a birth process (or Yule process) with a linear birth rate. The transition probabilities included uncertainties in detecting and accurate sizing of defects. Optimal inspection schedules were then determined while attempting to minimise the probabilities of time to failure before inspections and before the end of the pipe service life.

Critique. These models like those developed by Ahammed and Melchers (1994, 1995) are suited for ductile pipe materials since they use the description of residual strength suggested by Keifner and Vieth (1989). Stresses are considered in the in-plane direction only, which is a limitation for broad application to water mains, especially those with smaller diameters. However, corrosion-pitting rate is not modelled explicitly but the models are posed to take advantage of periodic inspections of oil and gas pipelines using non-destructive evaluation techniques. This approach although costly, may be preferable since reliable prediction of growth of corrosion pits is a significant challenge and involves considerable uncertainties.

Data requirements. The models require the current pipe age, wall thickness and depths of internal and external corrosion pits, and lengths and their respective probability distributions to estimate the probability of failure of each pipe or pipe section. As previously noted, active use of non-destructive techniques to measure corrosion pit depths in the oil and gas industry has received widespread use because of the economics and existing regulations to meet a specified level of safety standards.

Model description. UtilNets is a decision-support system for rehabilitation planing and optimisation of buried grey cast-iron water mains, which is currently being developed under the sponsorship of the European Union. According to Hadzilacos et al. (2000), the system performs reliability-based life predictions of the pipes and determines the consequences of maintenance and neglect over time in order to optimise rehabilitation policy. The system gives a probabilistic measure of the likelihood of structural, hydraulic, water quality and service failure of pipe segments and of the entire distribution system. Specific descriptions of the UtilNets model are not publicly available and hence it is not possible to offer a critique or specific data requirements. However, based on publications co-authored by the principal developer of the system, e.g., Camarinopoulos et al. (1996a, 1996b, 1999), the following appears to be relevant to physical modelling of structural behaviour of pipes.

The authors based their approach on mechanical models similar to those of Rajani and Makar (1999). They considered factors such as frost load (as a fraction of earth load), earth load, traffic load, pipe working pressure and surge pressure, temperature change (not clear if ambient or water temperature), internal and external pipe diameter, bursting strength, tensile strength, internal and external corrosion coefficients, unsupported length of pipe (causing beam condition), maximum external load and fracture toughness. They used a simple power function to model corrosion rates, which is similar to the approach used by Kumar et al. (1984) and Ahammed and Melchers (1994). They applied these physical/mechanical models in a probabilistic framework, to compute the reliability of a water main as the probability of its survival over a pre-defined time period. They reportedly developed a computer code in which they used a combination of Monte-Carlo simulations and approximate quadrature analytical methods to compute the results numerically. The authors did not provide any information about the availability of data for the validation of their model.

From the information described above, it appears that the mechanical/physically based model of UtilNets is close to a probabilistic version of the approach taken by Rajani and Makar (1999), which is illustrated in Figure 4.

Statistical Treatment of Temperature Effects

The effects of temperature on pipe breakage rates have been observed and reported by many. Walski and Pelliccia (1982) suggested that pipe breakage rates might be correlated to the maximum frost penetration in a given year. To account for the lack of frost penetration data, they correlated annual breakage rates with the air temperature of the coldest month, using a multiple regression analysis with age and air temperature as the covariates

$$N(t,T) = N(t_0)e^{At}e^{BT} \tag{11}$$

where t = pipe age; $N(t_0)$ = breaks per km at year of installation; T = average air temperature in the coldest month; A, B = constants. The authors did not provide any information as to the quality of breakage predictions that were obtained by this model.

Newport (1981) analysed circumferential pipe breakage data from various areas of the Severn-Trent Water Authority. He found that increased breakage rates coincided with cumulative degrees-frost (usually referred to as freezing index in North America and expressed as degree-days) in the winter as well as with very dry weather in the summer. He attributed the increase in winter breakage rates to the increase in earth loads due to frost penetration, i.e., frost loads, and the summer breakage rates to the increase in shear stress exerted on the pipe by soil shrinkage in a dry summer. He also observed that when two consecutive cold periods occurred, the breakage rates (in terms of breaks per degree-frost) in the first one exceeded those of the second one. He rationalised that the early frost “purged” the system of its weakest pipes, causing the later frost to encounter a more robust system. Newport (1981) used data from years 1970-1977 to obtain the following linear relationship (correlation coefficient of 0.9) between the number of water main breaks and the cumulative degrees-frost in a given year

$$\text{Total bursts per year} = 2.5\ (\text{Total degrees of frost}) + 500 \quad (12)$$

Equation (12) was derived for the Soar division of the Severn Trent distribution system and length of water mains was not specified. This linear model suggests that every degree of frost is responsible for an additional 2.5 breaks but at the same time, only about 50% of the failures may be explained by cold climate, expressed as cumulative degrees-frost. No model to predict water main breaks during extreme dry weather was given.

Habibian (1994) analysed the distribution system of Washington (DC) Suburban Sanitary Commission and observed an increase in water main breakage rate as the temperatures dropped. He related the breakage rates to the water temperature at the system intake rather than to the ambient air temperature, reasoning that although their monthly averages are similar, ambient air temperatures display sharp fluctuations while water temperatures are better surrogates for underground pipe environment. He concluded that the water temperature drop, rather than the absolute water temperature, had a determining influence on the pipe breakage rate. He also observed that in a given winter, similar consecutive temperature drops did not necessarily result in similar breakage rates, however, typically every time the temperature reached a new low a surge occurred in the number of pipe breaks. His explanation for this phenomenon concurred with Newport (1981), namely, that every temperature low "purged" the system of its weakest pipes, thus a new low affected the pipes that were a little more robust than those that had broken in the previous cold spell. During the warm seasons the pipes continue to deteriorate and the process is repeated in the subsequent cold season. It should be noted that Habibian's (1994) observations were all based on one-year data. Lochbaum (1993) reported that a study conducted by Public Service Electric and Gas Co. which showed that the breakage rates of cast iron gas pipes increased exponentially with the number of degree-days (degree-days were defined as $S_{i?N}$ $[(65^oF-T_i]$ where T_i is the average temperature in ^oF in day i and N is the set of days in a given month with average temperature below 65^oF). Lochbaum (1993) did not present a model relating the number of water main breaks to monthly degree-days; her observations, however, agreed with those of Newport (1981) and others. No information is provided as to the causes of pipe breaks in the warm seasons.

Sacluti et al. (1998) applied an artificial neural network (ANN) to the distribution system of a sub-division in Edmonton, Canada. The ANN model was applied to the entire network as a single entity (rather than to individual pipes) and was trained with data that included temperature (water and ambient), rainfall, operating pressure and historical data on break numbers. Since the model considered an entire network as a single entity, variates such as pipe age, type and diameter could not be considered, as well as geographical variates such as soil properties. The network consisted of spun-cast 150-mm (6") water mains. A sensitivity analysis determined that rainfall and operating pressure did not contribute to the predictive power of the model were thus omitted. The authors claim that the model was successfully applied to a holdout sample, demonstrating that the ANN "learnt" the breakage patterns rather than memorised them[3].

The ANN model was applied to a relatively small network with water mains that were relatively homogeneous with respect to type of pipe and operational and environmental conditions. A more heterogeneous set of water mains would likely require more data. The model predicted the number of water main breaks based on a 7-day weather forecast. This requirement limited its applicability to short term response rather than its use for long term planning purposes. The uncertainty in the weather forecast models also has a direct influence in the anticipated short-term response required. In its present form, the model can only be applied to homogeneous groups of water mains, for short-term planning of the maintenance work force required during an anticipated cold spell.

The ANN can be a useful tool in predicting water main breaks. Its main strength is that complex physically based models need not be developed and understood in order to identify breakage patterns. In that, it is similar to the various probabilistic models. Additional advantages are that many variates can be easily considered (many variates cause the probabilistic models to become computationally complex) and that software programs are readily available. Its weakness is that many types of variates have to be considered initially because the underlying physically based model is not known and often the available data is insufficient. Further, several variates can act on the breakage rates multiplicatively, additively, exponentially or in any combination thereof. Without knowledge of the physical process the ANN model has to be investigated for many combinations of inputs. Finally, training and validation of the model could require a lot of data that is often unavailable.

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[3] In ANN there is always a concern that the model will be "over-trained" resulting in a model that is just capable of "memorising" the training data set rather than being able to generalise the patterns to new sets.

Summary

The distribution network is the single most expensive component of a water supply system. The deterioration of water mains results in high maintenance costs, reduction in water quality, reduction in quality of service and loss of water. Planning for water main rehabilitation and renewal is imperative to meet adequate water supply objectives. The ability to understand and quantify pipe deterioration mechanisms is an essential part of the planning procedure.

The physical/mechanical models attempt to predict pipe failure by analysing the loads to which the pipe is subject as well as the capacity of the pipe to resist these loads. The residual structural capacity of pipes exposed to material deterioration and subjected to external and internal loads requires the consideration of numerous components such as frost loads, influence of corrosion rates, and materials properties such as strength and fracture toughness, etc. This paper covered the available physical/mechanical models that lead to an improved understanding of the structural performance of water mains. The various statistical methods that have been proposed in the scientific literature to explain, quantify and predict pipe breakage or structural failures of water mains were summarised in the companion paper.

The physical/mechanical models to predict water main breakage events were described, critiqued and their data requirements were identified. Some of the models address only one component of the physical process leading to breakage, while others attempt to address the problem in a more comprehensive way. The models were classified as either deterministic or probabilistic, depending the approach taken to represent deterioration and failure processes. Table 1 provides a graphical summary of the classes of models, and a comparison of what components of the physical process are addressed by each model. The role of temperature on breakage rate and the process by which low temperatures lead to pipe structural failures were discussed.

There is little doubt that robust and comprehensive physically based modelling is the ultimate goal in failure prediction. A true physically based model would explicitly encompass all the inter-relations between the factors affecting pipe breakage, and eliminate the need to use statistics to identify breakage patterns (although statistics would still be needed to address variability in the data and in the parameters). However, even the best analysis is only as good as the data that are available for its implementation, and in the water supply industry many of these data are unavailable or very costly to acquire. It appears that currently, only large water mains with costly consequence of failure may justify the accumulation of data that are required for physically based model application. The statistical models seem to be an economically viable approach for the smaller distribution water mains. More research is still required for improving and validating both the statistical and the physically based models.

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Table 1. Components of physically based models.

Class of models	Reference	Model components (numbers refer to those given in Figure 4)									
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Probabilistic	Ahammed & Melchers (1994, 1995)	100%	100%	25%	50%	50%	0%	50%	50%	50%	50%
Probabilistic	Hong (1997, 1998); Pandey, Stephens (1994a,1994b); Linkens et al. (1998)	100%	100%	75%	0%	50%	50%	0%	50%	50%	50%
Deterministic	Rajani & Makar (1999)	100%	100%	100%	100%	100%	100%	75%	50%	75%	25%
Deterministic	Doleac et al.(1980); Kumar et al.(1984); Randall-Smith et al. (1992)	100%	100%	100%	75%	25%	0%	0%	0%	25%	0%

Qualitative scales for estimates of models that account for specific components: 100% 75% 50% 25% 0%

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