

Probabilistic failure analysis of underground flexible pipes

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Abstract. Methods for estimating structural reliability using probability ideas are well established. When the residual ultimate strength of a buried pipeline is exceeded the limit, breakage becomes imminent and the overall reliability of the pipe distribution network is reduced. This paper is concerned with estimating structural failure of underground flexible pipes due to corrosion induced excessive deflection, buckling, wall thrust and bending stress subject to externally applied loading. With changes of pipe wall thickness due to corrosion, the moment of inertia and the cross-sectional area of pipe wall are directly changed with time. Consequently, the chance of survival or the reliability of the pipe material is decreased over time. One numerical example has been presented for a buried steel pipe to predict the probability of failure using Hasofer-Lind and Rackwitz-Fiessler algorithm and Monte Carlo simulation. Then the parametric study and sensitivity analysis have been conducted on the reliability of pipeline with different influencing factors, e.g. pipe thickness, diameter, backfill height etc.

Keywords: probability of failure; flexible pipes; Hasofer-Lind and Rackwitz-Fiessler algorithm; Monte Carlo simulation; deflection; buckling; wall thrust; bending stress

1. Introduction

Failure management of buried pipeline systems is one of the important issues for asset managers. A large section of pipes in the UK are classified as structurally deficient and functionally obsolete. Every year billions of pounds are spent by water industries for their maintenance and renewal. Due to their low visibility, condition assessment and rehabilitation of the underground pipelines are frequently neglected. Lack of detailed knowledge on the condition of pipelines escalates vulnerability to catastrophic failures (Chughtai and Zayed 2008). The behaviour of buried pipes is considerably influenced by uncertainties due to external loads, pipe materials and surrounding soil properties etc. The decision to repair or replace current pipe is typically based on performance indicators such as structural integrity, hydraulic efficiency and system reliability (Rajani and Makar 2000). Ahammed and Melchers (1994) stated that the pipe replacement or rehabilitation is typically determined by the performance parameter, i.e. the annual number of failure in a given section of pipe networks. This approach depends largely on what has happened in the past and what is expected to happen in the future.

A complicated approach to scheduling pipe maintenance is to determine individual pipes that are approaching unsafe condition and repair or replace them before they fail. This type of approach

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requires a robust methodology to determine the remaining safe service life of each pipe segment within the distribution network system. Many challenges have faced by water industry during placing or maintenance the underground pipeline networks. The most common challenges are found as buckling, deflection, wall thrust and bending behaviour of pipe under hydrostatic pressure, poor design detailing and installation practices during placing the pipes, insufficient corrosion protection procedures, pipe material deterioration, scouring underneath the ground level, frost heave action and insufficient understanding of the product limitations. In reality, a buried pipe's mechanical strength begins to decrease as soon as it is installed because of the environmental conditions surrounding the pipe. Due to their low visibility and lack of proper information regarding underground pipes' condition, assessment and maintenance are frequently neglected until a disastrous failure occurs. The long-term planning of the renewal of underground pipe distribution networks requires the ability to predict system reliability as well as assess the economic impact (Tee and Li 2011).

For buried pipelines subject to both internal and external loading, a vital failure criterion is the loss of structural strength which is influenced by localized or overall reduction in pipe wall thickness (Ahmed and Melchers 1997). The size of the resulting thickness undermines the pipe resistance capacity which in turn reduces the factor of safety of the whole distribution system. In the past, various researchers and organizations recognized the importance and the applicability of probabilistic approach in the reliability estimation of buried flexible pipeline systems. The development of reliability analysis in pipe systems is receiving considerable attention nowadays and the guidelines on the targeted reliability indices have been suggested recently. Methods of reliability analysis such as first order reliability method (FORM), second-order reliability method (SORM), point estimate method, and Monte Carlo simulation are available in literature (Baecher and Christian 2003). Complex models in risk and reliability analysis often involve uncertain input parameters which can be determined using these methods with varying degrees of accuracy. These parameters are best explained by random variables with known or assumed probability distributions.

Babu and Rao (2005) conducted reliability analysis on flexible buried steel pipe in terms of two failure modes, namely, deflection and buckling. Later, Babu *et al.* (2006) predicted the underground flexible pipe failure due to deflection, buckling and wall thrust. These analyses were based on pipe installation time only, i.e., non-time dependent. Hence, the practical motivation of this study is to investigate these different failure modes with respect to time. In reality, underground pipeline failure is a continuous and time-variant process. The strength of buried metal pipe decreases due to corrosion, fatigue and overloading and this affects the pipe reliability over time. As a result, the safety or serviceability margin and the corresponding reliability index decreases (or failure probability increases) with respect to time. Since the life of an underground pipeline is related to corrosion deterioration and the developed stresses, the available design equations (ASCE 2001, Gabriel 2011) for pipeline as well as pipeline thickness loss due to corrosion are used for the purpose of establishing a probabilistic functional relationship between loads, corrosion, related soil and material random variables. In the probabilistic approach, the input parameters are treated as continuous random variables and the performance of the structure resulting from different failure criteria is expressed in a probabilistic pipeline framework, i.e., either probability of failure or in terms of reliability index. The key component of this methodology is to estimate the probability of failure to predict the expected residual safe service life of pipes. Therefore, this paper focuses on the results of the analysis of the buried flexible pipes conducted in the light of the aforementioned points and has the following objectives:

(a) The failure probability analysis of buried flexible pipe system from different failure criteria, i.e., excessive deflection, buckling, wall thrust, bending stress and strain based on the available conventional design equations.

(b) Comparative study of safety in terms of probability of failure or survival time for flexible pipes in different failure modes and parametric analysis for different influencing factors.

2. Formulation for prediction of pipe failure

2.1 Corrosion of flexible steel pipes

Buried pipes are made of plastic, concrete or metal, e.g. steel, galvanized steel, ductile iron, cast iron or copper. Plastic pipes tend to be resistant to corrosion. Damage in concrete pipes can be attributed to biogenous sulphuric acid attack (Tee *et al.* 2011). On the other hand, metal pipes are susceptible to corrosion. Metal pipe corrosion is a continuous and variable process. Under certain environmental conditions, metal pipes can become corroded based on the properties of the pipe, the soil surrounding the pipe wall, water or wastewater properties and stray electric currents. The corrosion pit depth D_T can be modelled with respect to time as follows (Ahammed and Melchers 1994, Sadiq *et al.* 2004)

$$D_T = kT^n \quad (1)$$

where k is multiplying constant, n is exponential constant and T is exposure time. For simplification, the corrosion pit depth is considered as uniform around the entire circumference of the pipe and based on this assumption, moment of inertia, I and cross-sectional area of pipe wall per unit length, A_s are shown in Eqs. (2) and (3) respectively (Watkins and Anderson 2000)

$$I = (t - D_T)^3 / 12 \quad (2)$$

$$A_s = t - D_T \quad (3)$$

In this study, D_T , I and A_s are time dependent variables. The corrosion parameters k and n are highly uncertain and therefore are treated as random variables in the reliability analysis. These values are typically determined from regression analysis using observed and experimental data obtained for specific soil and other conditions. When there is little or no information on which to base a choice for corrosion constant parameters, then known values from other situations might be used, with judicious selection of means and variances (Ahammed and Melchers 1994).

2.2 Flexible pipe failure modes

Pipe behaviour can be broadly classified as flexible or rigid, depending on how it performs when installed. All types of underground pipes whether flexible or rigid, rely on the backfill properties to transfer the loads into the bedding. Structures collapse when the applied stress exceeds the limiting or ultimate strength of the pipe wall material. In this paper, the dominating failure criteria of flexible pipes are characterised by limit states as follows

- (a) Excessive deflection;
- (b) Actual buckling pressure greater than the critical buckling pressure;
- (c) Actual wall thrust greater than critical wall thrust;

(d) Actual bending stress greater than the allowable stress.

2.2.1 Deflection

The performance of flexible pipe in its ability to support load is typically assessed by measuring the deflection from its initial shape. Deflection can be defined as the change in inside diameter that results when a load is applied to a flexible pipe. Deflection is quantified in terms of the ratio of the horizontal increase in diameter (or vertical decrease in diameter) to the pipe diameter. Rigid pipe is sometimes classified as pipe that cannot deflect more than 2% without significant structural distress, such as cracking. Flexible pipe takes advantages of its ability to move, or deflect under loads without structural damage. The critical deflection for flexible pipe, Δy_{cr} is determined as 5% of inside diameter of pipe (Gabriel 2011). According to BS EN 1295:1 (1997) and BS 9295 (2010) the deflection, Δy can be calculated as below

$$\Delta y = \frac{K(D_L W_c + P_s)D}{\left(\frac{8EI}{D^3} + 0.061E_s\right)} \quad (4)$$

where K is deflection coefficient, D_L is deflection lag factor, D is mean diameter, E is modulus of elasticity of pipe material and E_s is modulus of soil reaction. The deflection coefficient reflects the degree of support provided by the soil which is based on the type of installation. The loads acting on the pipe are governed by the term $D_L W_c + P_s$ where W_c is soil load and P_s is live load as follows

$$W_c = \gamma_s H \quad (5)$$

$$P_s = \frac{W_s I_f}{L_1 L_2} \quad (6)$$

where γ_s is unit weight of the soil, H is height of soil on the top of pipe, W_s is wheel load, I_f is impact factor with the value of 1.1 for $0.6 \text{ m} < H < 0.9 \text{ m}$, or 1 for $H \geq 0.9 \text{ m}$, L_1 and L_2 are load width parallel and perpendicular to direction of travel, respectively (Sarplast 2008).

2.2.2 Buckling

External loadings from soil pressure or external hydrostatic pressure can cause inward deformation known as wall buckling. Buckling is a premature failure in which the pipe is not able to maintain its initial circular shape when the tangential compressive stress reaches a limit value and the pipe distorts unstably in buckling. Wall buckling can occur due to insufficient pipe stiffness. The more flexible the pipe, the more unstable the wall structure will be in resisting wall buckling. Pipe encased in soil may buckle due to excessive loads and deformations.

The two buckling forms are usually called general (global) buckling and local buckling (Berti *et al.* 1998). Global buckling is a load response mode due to compressive effective axial force but not a failure mode. Pipe is safe from buckle globally whereas local buckling is an important limit state. The total load or the actual buckling pressure must be less than the critical buckling pressure for the safety of the structure. The actual buckling pressure for flexible pipe, p can be calculated as follows (AWWA 1999)

$$p = R_w \gamma_s + \gamma_w H_w + P_s \quad (7)$$

where R_w is water buoyancy factor, γ_w is unit weight of water, H_w is height of groundwater above pipe springline and P_s is live load as calculated in Eq. (6). The critical buckling pressure p_{cr} is calculated as follows (AWWA 1999)

$$p_{cr} = \frac{1}{S_f} \sqrt{\left(32 R_w B' E_s \frac{EI}{D^3} \right)} \quad (8)$$

where S_f is safety factor, E_s is soil modulus and B' is empirical coefficient of elastic support. The critical buckling pressure depends on the surrounding soil pressure. The critical buckling pressure increases/decreases as the effective soil pressure surrounding the pipe increases/decreases.

2.2.3 Wall thrust

Wall thrust or wall crushing is characterised by localised yielding when the in-wall stress reaches the yield stress of the pipe material. If the buried depth is not enough then the pipe wall can crush due to earth and surface loading. Buried depth should be sufficient to avoid the crushing of the side wall. Thrust or stress on the pipe wall is the total load on the pipe wall including soil loads, traffic loads and hydrostatic loads. If only dead load is involved during the installation, the long-term material properties are considered throughout the calculation of wall thrust analysis. However, if both dead load and live load are present (typically any vehicular load with 2.4 m of cover or less), two wall thrust analyses are required (Gabriel 2011) as follows

- (a) Accounts both dead load and live load and employs the short term material properties throughout the procedure;
- (b) Accounts only dead load and employs the long-term material properties throughout the process.

Then, the more conservative limit state function value is used for wall thrust analysis. Note that some flexible pipes have both short-term and long-term material properties (elasticity of modulus), where short-term value is higher than long-term value, such as, polyethylene pipe and therefore, two wall thrust analyses are applicable but some have no such classification, for example, ductile iron pipes, steel pipes etc. and for these cases, only above option a) is considered for analysis, where short-term and long-term properties are the same.

The critical and actual wall thrust can be estimated by applying Eq. (9) and Eq. (10), respectively (Gabriel 2011). The allowable or critical thrust must be equal to or greater than the actual thrust in order to remain structurally stable.

The critical wall thrust

$$T_{cr} = F_y A_s \phi_p \quad (9)$$

where F_y is the minimum tensile strength of pipe, A_s is cross-sectional area of pipe wall per unit length and ϕ_p is capacity modification factor for pipe.

The actual wall thrust

$$T = 1.3(1.5W_A + 1.67P_s C_L + P_w)(D_o / 2) \quad (10)$$

where D_o is the outside diameter and C_L is live load distribution coefficient. The loads acting on the pipe considered in wall thrust analysis are soil arch load W_A , live load P_s and hydrostatic

pressure P_w which can be calculated as follows

$$W_A = P_{sp} V_{AF} \quad (11)$$

$$P_w = \gamma_w H_g \quad (12)$$

where P_{sp} is geostatic load, V_{AF} is vertical arching factor and H_g is height of groundwater above springline of pipe.

2.2.4 Bending stress and bending strain

The moment curvature relationship provides information necessary for pipe design against failure due to bending. If a pipe is part of a carrying structure, the elastic limit may be an obvious choice as the design limit. A pipe subjected to increasing pure bending will fail as a result of increased ovalisation of the cross section and reduced slope in the stress-strain curve. Up to a certain level of ovalisation, the decrease in moment of inertia will be counterbalanced by increased pipe wall stresses due to strain hardening. When the loss in moment of inertia can no longer be compensated for by the strain hardening, the moment capacity has been reached and catastrophic cross sectional collapse will occur if additional bending is applied (Hauch and Bai 1999).

For the safety of the pipe, the bending stress should not exceed the tensile strength of the pipe material and the longitudinal bending strain should not exceed the allowable strain limit of pipe materials. Therefore, checking the bending stress and strain is important to ensure that these are within material capability. Bending stress σ_b and bending strain ε_b can be calculated as follows (Gabriel 2011)

$$\sigma_b = 2D_f E \Delta_y y_0 S_f / D^2 \quad (13)$$

$$\varepsilon_b = 2D_f \Delta_y y_0 S_f / D^2 \quad (14)$$

where D_f is shape factor, y_0 is distance from centroid of pipe wall to the furthest surface of the pipe.

3. Probabilistic reliability analysis

Monte Carlo simulation (MCS) and Hasofer-Lind and Rackwitz-Fiessler (HLRF) algorithm are used in this study for estimating structural reliability of underground flexible pipes. Pipe failure can be defined in relation to different possible mechanisms as mentioned in Section 2. The limit state functions for the failure modes are defined as follows

$$Z(X) = \Delta y_{cr} - \Delta y \text{ (for deflection)} \quad (15)$$

$$Z(X) = p_{cr} - p \text{ (for buckling)} \quad (16)$$

$$Z(X) = T_{cr} - T \text{ (for wall thrust)} \quad (17)$$

$$Z(X) = \sigma_{cr} - \sigma_b \text{ (for bending stress) or } Z(X) = \varepsilon_{cr} - \varepsilon_b \text{ (for bending strain)} \quad (18)$$

where Δy_{cr} , p_{cr} , T_{cr} , σ_{cr} and ε_{cr} are critical pipe deflection, buckling, wall thrust, bending stress and

bending strain, respectively whereas Δy , p , T , σ_b and ε_b are actual pipe deflection, buckling, wall thrust, bending stress and bending strain, respectively.

3.1 Hasofer-Lind and Rackwitz-Fiessler (HLRF) algorithm

The primary benefit of HLRF is believed to lie in accuracy comparable with other rigorous techniques such as Monte Carlo simulation while requiring significantly less computational effort. Another benefit to HLRF over FORM is to evaluate a limit state function at a point known as design point or most probable point x_i^* , $i = 1, 2, \dots, n$ instead of mean value. The design point is a point on the failure surface $Z = 0$. Since this design point is generally not known a priori, an iteration technique must be used to solve for reliability index (Section 3.1.1). Pipe reliability has been estimated using HLRF algorithm where all the variables should be normally distributed. For non-normally distributed variable, Rackwitz-Fiessler (RF) algorithm is applied to transform it to a normally distributed variable. Mathematically RF techniques can be expressed as below (Haldar and Mahadevan 2000)

$$F_x(x^*) = \Phi\left(\frac{x^* - \mu_x^e}{\sigma_x^e}\right) \quad (19)$$

$$f_x(x^*) = \frac{1}{\sigma_x^e} \phi\left(\frac{x^* - \mu_x^e}{\sigma_x^e}\right) \quad (20)$$

where F_x is cumulative distributed function (CDF), f_x is probability density function (PDF), Φ is the CDF for the standard normal distribution and ϕ is the PDF for the standard normal distribution, μ_x^e is the equivalent normal mean and σ_x^e is the equivalent standard deviation. By manipulating Eqs. (19) and (20), the expressions for μ_x^e and σ_x^e are obtained as follows (Haldar and Mahadevan 2000)

$$\mu_x^e = x^* - \sigma_x^e \left[\Phi^{-1}(F_x(x^*)) \right] \quad (21)$$

$$\sigma_x^e = \frac{1}{f_x(x^*)} \phi\left(\frac{x^* - \mu_x^e}{\sigma_x^e}\right) = \frac{1}{f_x(x^*)} \phi\left[\Phi^{-1}(F_x(x^*))\right] \quad (22)$$

Limit state functions are defined as the difference between actual and critical values, such that $Z(X) < 0$ represents the failure state, $Z(X) > 0$ indicates a safe state, and the limit state boundary which separates the safety and failure domains, exists at $Z(X) = 0$. The probability of failure for each limit state function can be evaluated by

$$P_f = P[Z(X) < 0] = \Phi(-\beta) \quad (23)$$

where β is known as the safety index or reliability index.

Each limit state function in Eq. (23) is used to calculate the probability of failure of underground flexible pipe due to each failure mode. Eq. (23) implies that when β increases, the probability magnitude decreases. To evaluate the relative contribution of each random variable in the limit

state function $Z(X)$, sensitivity coefficient $\alpha_{x_i}^2$ can be calculated as follows

$$\alpha_{x_i}^2 = \frac{\left[\left(\frac{\partial Z}{\partial x_i} \right)^* \sigma_x^e \right]^2}{\sum_{i=1}^n \left(\frac{\partial Z}{\partial x_i} \sigma_x^e \right)^{*2}} \quad (24)$$

3.1.1 Procedure

HLRF algorithm is described briefly as follows

Step 1: Define an appropriate limit state equation.

Step 2: Assume an initial value of the safety index β . Any value of β can be assumed. If it is chosen intelligently, the algorithm will converge in a very few steps. An initial β value of 3.0 is reasonable.

Step 3: Assume the initial values of the design point x_i^* . In the absence of any other information, the initial design point can be assumed to be at the mean values of the random variables.

Step 4: Compute the mean and standard deviation at the design point of the equivalent normal distribution. Apply Rackwitz-Fiessler algorithm for those variables that are non-normal.

Step 5: Compute partial derivatives $(\partial Z / \partial x_i)^*$ evaluated at the design point x_i^* .

Step 6: Compute the direction cosines α_i at the design point as

$$\alpha_{x_i} = \frac{\left[\left(\frac{\partial Z}{\partial x_i} \right)^* \sigma_x^e \right]}{\sqrt{\sum_{i=1}^n \left(\frac{\partial Z}{\partial x_i} \sigma_x^e \right)^{*2}}}$$

If the random variables are normal, then their standard deviations can be used directly; otherwise, for non-normal random variables, the equivalent standard deviations at the design point need to be used.

Step 7: Compute the new values for design point x_i^* as

$$x_i^* = \mu_x^e - \alpha_i \beta \sigma_x^e$$

If necessary, repeat steps 4 through 7 until the estimates of α_i converge to a predetermined tolerance. A tolerance level of 0.001 - 0.005 is common.

Step 8: Compute an updated value for β using the condition that limit state equation must be satisfied at the new design point.

Step 9: Repeat steps 3 through 8 until β converges to a predetermined tolerance level.

Step 10: Calculate the failure probability P_f using Eq. (23).

3.2 Monte Carlo simulation (MCS)

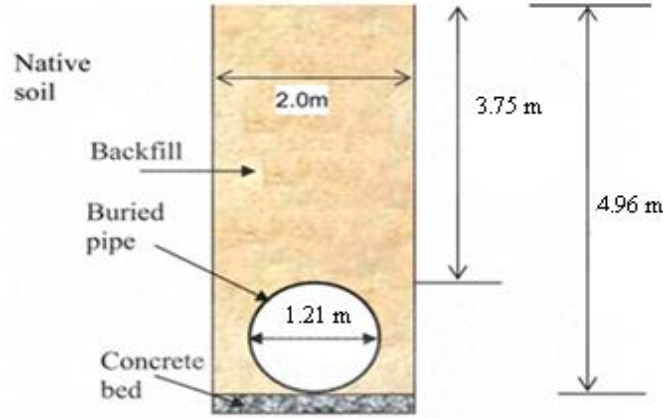


Fig. 1 Geometrical details of the buried flexible pipe section

The probability of failure, P_f is expressed by Eq. (23) or the following expression

$$P_f = P[Z(X_1, X_2, \dots, X_n) \leq 0] \quad (25)$$

where X_1, X_2, \dots, X_n are random variables. The Monte Carlo simulation allows the determination of an estimate of the probability of failure, given by

$$P_f = \frac{1}{N} \sum_{i=1}^N I(X_1, X_2, \dots, X_n) \quad (26)$$

where $I(X_1, X_2, \dots, X_n)$ is a function defined by

$$I(X_1, X_2, \dots, X_n) = \begin{cases} 1 & \text{if } g(X_1, X_2, \dots, X_n) \leq 0 \\ 0 & \text{if } g(X_1, X_2, \dots, X_n) > 0 \end{cases} \quad (27)$$

According to Eq. (26), N independent sets of values X_1, X_2, \dots, X_n are generated based on the probability distribution of each random variable and the failure function is computed for each sample. Using MCS, the probability of structural failure is estimated as follows

$$P_f = \frac{N_H}{N} \quad (28)$$

where N_H is the total number of cases where failure has occurred.

4. Numerical studies

An underground steel pipe with a mean diameter of 1.21 m and initial wall thickness of 21 mm under a roadway subjected to hypothetical operating conditions is taken as a numerical example. Calculations are presented for the case of a typical pipe section, as shown in Fig. 1. The pipe is circular and buried in a trench of 2 m width and 3.75 m depth. The backfill material has a unit

Table 1 Properties and parameters

Symbol description	Value
Buoyancy factor, R_w	1.00
Trench width, B_d	2.00 m
Outside pipe diameter, D_o	1.231 m
Inside pipe diameter, D_I	1.189 m
x-sectional area of pipe wall per unit length, A_s	0.021 m ² /m
Shape factor, D_f	4.0
Capacity modification factor for pipe, ϕ_p	1.00
Safety factor for bending, S_f	1.5
Tensile strength of pipe, F_y	450 MPa
Safety factor for Buckling, S_f	2.5
Poisson Ratio, ν	0.3
Allowable Strain, ϵ_{cr}	0.2%

Table 2 Statistical properties of random variables

Material properties	Mean (μ)	COV* (%)	Distribution
Elastic modulus of pipe, E	213.74×10 ⁶ kPa	1.0	Normal
Backfill soil modulus, E_s	10 ³ kPa	5.0	Normal
Unit of weight of soil, γ_s	18.0kN/m ³	2.5	Normal
Wheel load (Live load), P_s	80.0 kPa	3.0	Normal
Deflection coefficient, K	0.11	1.0	Lognormal
Multiplying constant, k	2.0	10.0	Normal
Exponential constant, n	0.3	5.0	Normal
Thickness of pipe, t	0.021 m	1.0	Normal
Height of the backfill, H	3.75 m	1.0	Normal

*COV = Coefficient of Variation

weight of 18.0 kN/m³ and soil modulus of 10³ kPa. The buried pipe is considered under a heavy traffic condition where the wheel load is 80 kPa. Other parameters are listed in Table 1. There are 9 random variables where the mean and coefficient of variation are listed in Table 2. The random variables are deflection coefficient (K), soil density (backfill) (γ_s), elasticity of pipe material (E), height of soil above pipe (H), pipe wall thickness (t), multiplying constant (k), exponential constant (n), live load (wheel load) (P_s) and soil modulus (E_s). The mean values along with the coefficients of variation for the theses parameters are used in the calculations. All of them are considered as normally distributed, except the deflection coefficient which is log-normal distributed. Thus Rackwitz-Fiessler algorithm has been applied to transform its distribution from log-normal to normal in this study. The pipe is subjected to corrosion and its corrosion rate is modelled using Eq. (1). Assuming the change of pipe surface due to corrosion is uniform over the entire surface area.

4.1 Probability of failure

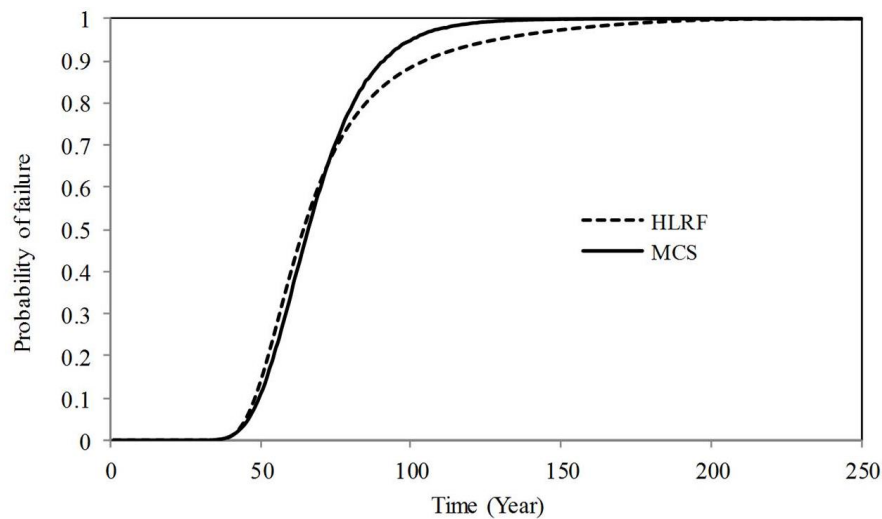


Fig. 2 Probability of failure for limit state due to corrosion induced deflection using HLRF and Monte Carlo simulation

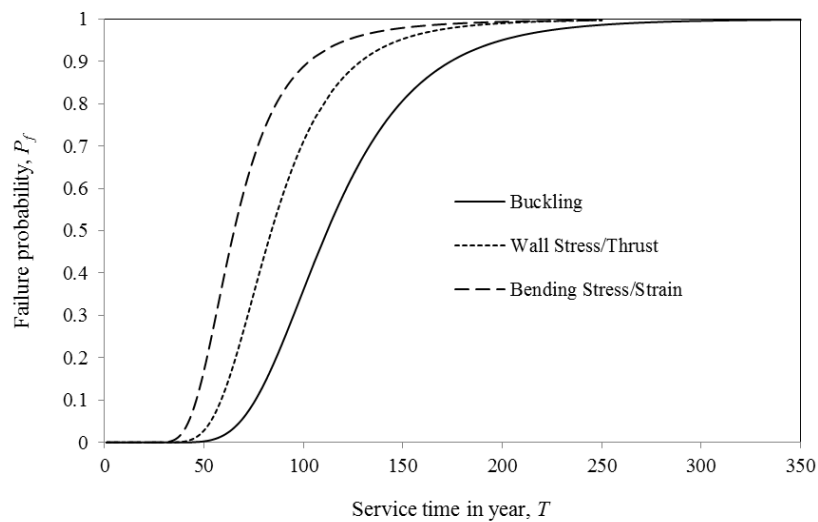


Fig. 3 Probability of failure for limit states due to corrosion induced buckling, excessive wall thrust and bending stress

In the case of buried pipes the assessment of probability of failure on year basis is useful which enables calculation of reliability over time. The probabilities of failure for corrosion induced excessive deflection, buckling, wall thrust and bending stress with respect to time have been estimated using the material properties and random variables which are presented in Tables 1 and 2. First, the probability of failure due to corrosion induced deflection is calculated using HLRF algorithm and MCS (with 10^6 samples). The results from both methods are in reasonable agreement as shown in Fig. 2.

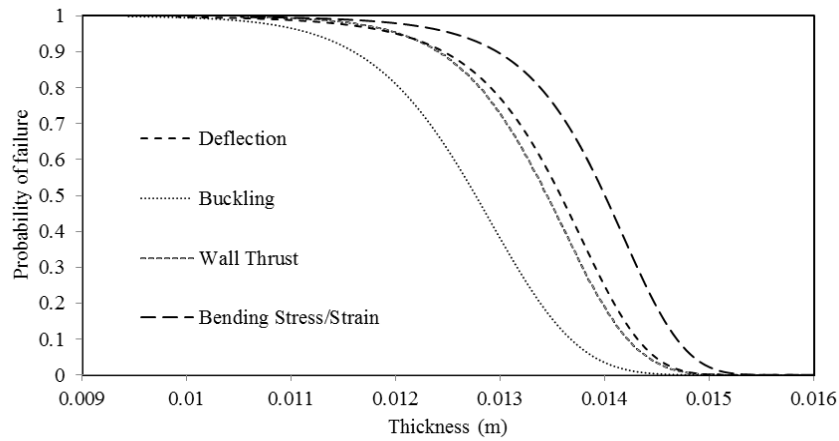


Fig. 4 Probability of failure for different limit states versus pipe wall thickness

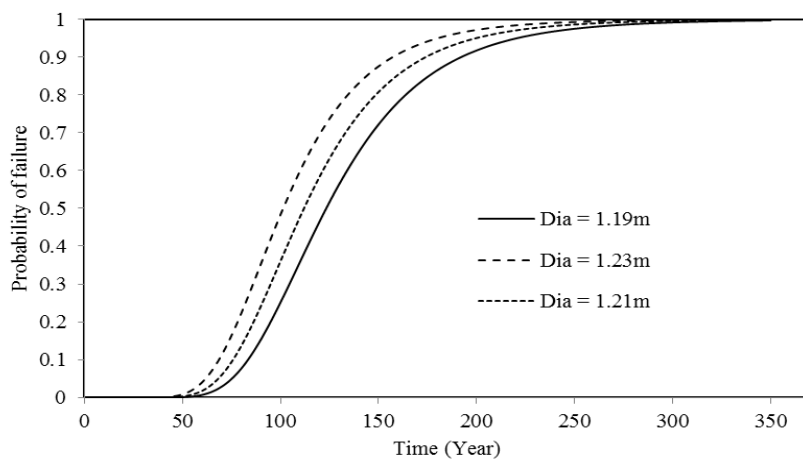


Fig. 5 Probability of failure for different pipe diameters due to corrosion induced buckling

Next, only Monte Carlo simulation is used for estimation of failure probability for ultimate limit states due to corrosion induced buckling, excessive wall thrust and bending stress. Fig. 3 shows that excessive bending stress is the most critical failure event whereas buckling has the lowest probability of failure during the whole service life of the pipe. Considering the failure probability of 0.1 (10%) as a threshold level for the safe service life (Babu and Srivastava 2010), the study illustrates that the safe service life in the worst case scenario is less than 50 years.

4.2 Parametric study

To analyse the effect of the design variables on the probability of failure of the underground flexible pipeline system, a parametric study has been carried out. The effect of changing the wall thickness, pipe diameter and backfill height on the probability of failure has shown in Figs. 4, 5 and 6, respectively.

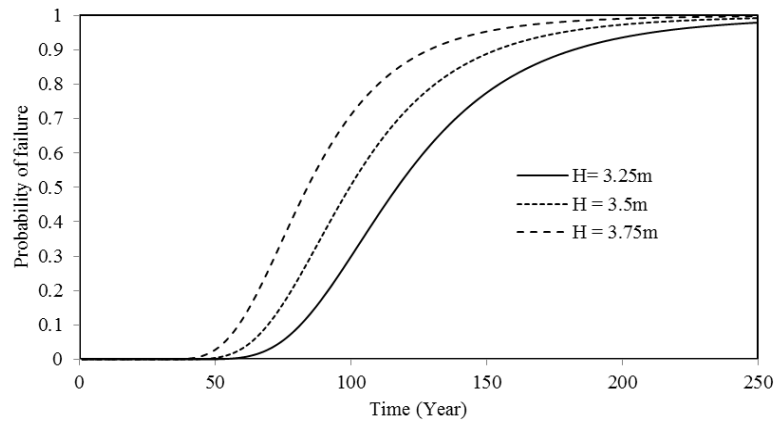


Fig. 6 Probability of failure for different backfill heights due to corrosion induced excessive wall thrust

4.2.1 Pipe wall thickness

The original thickness of the pipe is 21 mm but the range of the x-axis of the Fig. 4 (pipe wall thickness) is plotted from 16 mm because pipe failure does not commence (probability of failure is almost equal to zero) in the beginning of the pipe lessening process for all the corrosion induced failure criteria until the residual thickness reaches 15.5 mm. On the other hands, the pipe totally fails (probability of failure is almost equal to one) when the residual thickness reaches 12 mm for all the failure criteria except the failure due to buckling. It is observed that in the case of failure due to buckling, the result shows that the pipe needs more thickness reduction to fail than others. In this case, the range is from 14.5 mm to 10.5 mm. Therefore, it can be concluded that buckling is the least susceptible whereas the excessive bending stress is the most susceptible due to reduction of the pipe wall thickness.

4.2.2 Pipe diameter

The probability of failure has been estimated for pipe diameter of 1.19 m, 1.21 m and 1.23 m using all the four different failure criteria. However, only result for buckling is shown in this paper (Figure 5) because it shows the largest effect due to changes in pipe diameter. In fact, the results show that the larger diameter pipes have a higher failure rate than the smaller ones i.e. the service life of pipe decreases as pipe diameter increases. The obtained results are in a good agreement with the results of the previous studies. A number of authors have investigated the relationship between sewer size and its structural reliability. Involving large sewer samples with length of 180 km, O'Reilly *et al.* (1989) noticed that the incidence of longitudinal cracks increased with diameter and also observed that fractures were much more common in larger size of pipes. O'Reilly *et al.* (1989) also observed that when all defects were considered, the pipes with middle ranges of diameters (300 – 700 mm) showed more defects than those with smaller sizes. Larger pipes are more at risk to structural damage due to their bulk and weight making them more difficult to lie accurately (Davis *et al.* 2001). Similarly, this analysis also shows that the highest failure probability is due to excessive bending stress and the lowest is due to buckling.

4.2.3 Soil height

Like pipe diameter, backfill height also affects pipe failure rate. The probability of failure for

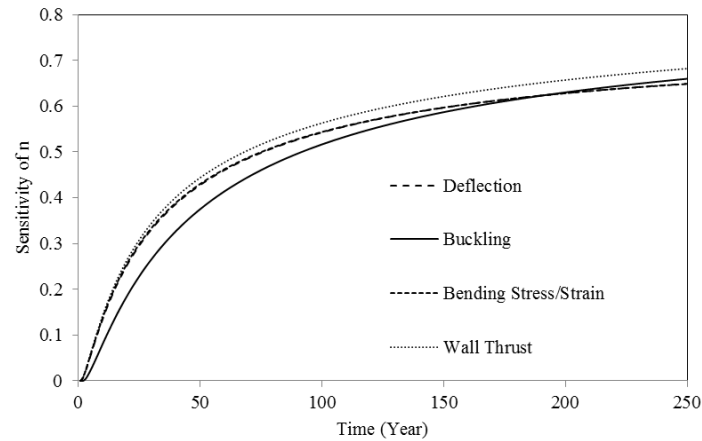


Fig. 7 Sensitivity of exponential constant for different limit states during pipe service life

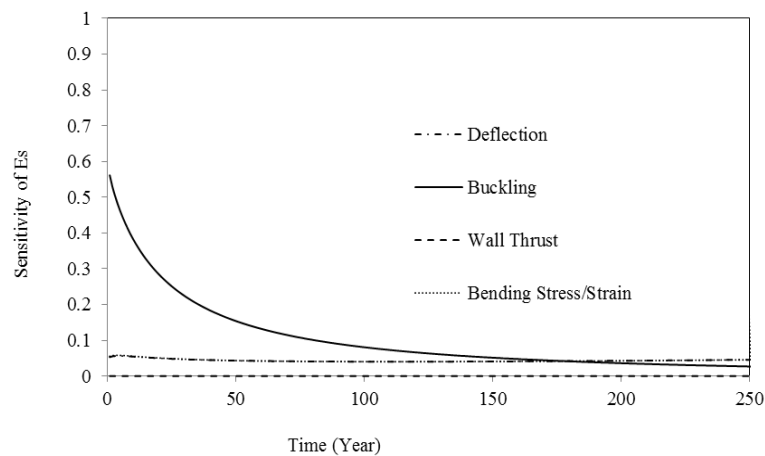


Fig. 8 Sensitivity of soil modulus for different limit states during pipe service life

every 250 mm change of soil height has been estimated i.e. soil height of 3.25 m, 3.50 m and 3.75 m using the above mentioned failure criteria. Again, only result for the failure case due to excessive wall thrust is shown in this paper (Fig. 6) because it shows the largest effect due to changes in soil height. This makes sense because soil height has the most influence on wall thrust compare to others. With increasing the backfill height, this will increase soil pressure or overburden pressure and further reduce the service life of the pipe. Thus, failure rate increases with increasing the soil height above the pipe invert. This leads to the same conclusion given by Davis *et al.* (2001) in investigating the effect of soil depth on sewer structural condition and reliability.

4.3 Sensitivity analysis

In addition to the parametric studies above, sensitivity analysis has been carried out to evaluate the relative contribution of each random variable (listed in Table 2) in the four limit state functions

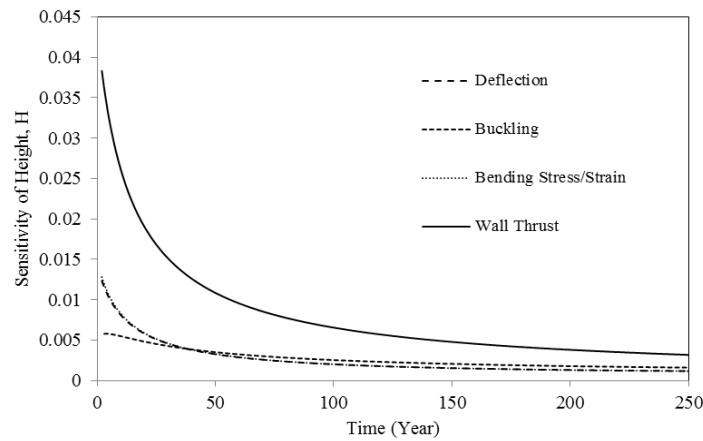


Fig. 9 Sensitivity of backfill height for different limit states during pipe service life

using Eq. (24). The analysis shows that soil (backfill) density, soil modulus, wheel load, multiplying constant, exponential constant and pipe thickness are among larger contributors and on the other hand, elastic modulus of pipe material, deflection coefficient and soil height above pipe invert have minor contribution to the pipe reliability. Selected results are shown in this paper due to space limitations that are exponential constant in Fig. 7, soil modulus in Fig. 8 and soil height in Fig. 9.

At the early stage of the pipe service life, the sensitivity of the exponential constant in Eq. (1) has zero impact and increases significantly with the pipe age for all the four failure criteria as shown in Fig. 7. This is attributed to the fact that corrosion does not cause any problem to new pipes but mainly the root cause of failure and collapse for aging pipes. The sensitivity analysis also shows that soil modulus has zero impact on the failure due to excessive wall thrust as shown in Figure 8 because it does not appear in Eqs. (9) and (10). In contrast, Fig. 9 shows that backfill height has the largest impact as predicted in the previous parametric study, although it only has minor contribution compare to other random variables.

5. Conclusions

A failure probability analysis of underground flexible pipeline has been presented in this paper using HLRF algorithm and Monte Carlo simulation. The behaviour of buried pipes is considerably influenced by uncertainties due to external loads, corrosion parameters, pipe materials and surrounding soil properties etc. The dominating failure criteria of flexible pipes are characterized by corrosion induced excessive deflection, buckling, excessive wall thrust and bending stress. The results show that excessive bending stress is the most critical failure event whereas buckling is the least susceptible during the whole service life of the pipe. In addition, parametric study and sensitivity analysis have been performed to analyse the effect of the design variables on the probability of failure of the underground flexible pipeline system. The proposed work is studied with independent failure events. More complicated and correlation between the failure events and random variables will be considered in the future work. The estimation of failure probability may

then be utilized to predict the maintenance and repair options during expected service life time and hence, a renewal strategy can be applied to avoid unexpected collapse or failure of the pipe network.

References

- Ahammed, M. and Melchers, R.E. (1994), "Reliability of underground pipelines subject to corrosion", *Journal of Transportation Engineering, ASCE*, **120**(6), 989-1002.
- Ahammed, M. and Melchers, R.E. (1997), "Probabilistic analysis of underground pipelines subject to combined stresses and corrosion", *Engineering Structures*, **19**(12), 988-994.
- ASCE (American Society of Civil Engineers) (2001), 'Guidelines for the design of buried steel pipe', American Lifelines Alliance, USA.
- AWWA (American Water Works Association) (1999), 'Fiberglass Pipe Design', *AWWA Manual M45*, USA, 35-53.
- ASCE (American Society of Civil Engineers) (2001), 'Guidelines for the design of buried steel pipe', *American Lifeline Alliance and ASCE*, USA, 9-20.
- Babu, S.G.L. and Srivastava, A. (2010), "Reliability analysis of buried flexible pipe-soil systems", *Journal of Pipeline Systems Engineering and Practice, ASCE*, **1**(1), 33-41.
- Babu, S.G.L., Srinivasa, M.B.R. and Rao, R.S. (2006), "Reliability analysis of deflection of buried flexible pipes", *Journal of Transport Engineering*, **132**(10), 829-836.
- Barbosa, M.R., Morris, D.V. and Sarma, S.V. (1989), "Factor of safety and probability of failure of rockfill embankments", *Geotechnique*, **39**(3), 471-483.
- Berti, D., Stutzman, R., Lindquist, E. and Eshghipour, M. (1998), "Technical forum: buckling of steel tunnel liner under external pressure", *J. Energy Eng.*, **124**(3), 55-89.
- BS EN 1295:1 (1997), 'Structural design of buried pipelines under various conditions of loading- General requirements', *British Standards Institution*, UK.
- BS 9295 (2010), 'Guide to the structural design of buried pipelines', *British Standards Institution*, UK.
- Baecher, G.B. and Christian, J.T. (2003), *Reliability and statistics in geotechnical engineering*, Wiley, New York, USA.
- Chughtai, F. and Zayed, T. (2008), "Infrastructure condition prediction models for sustainable sewer pipelines", *Journal of Performance of Constructed Facilities*, **22**(5), 333-341.
- CPSA (Concrete Pipeline Systems Association) (2008), Charles Street, Leicester, UK
- Farshad, M. (2006), *Plastic Pipe Systems: Failure Investigation and Diagnosis*, First Edition. Elsevier Ltd.
- Gabriel, L.H. (2011), "Corrugated polyethylene pipe design manual and installation guide", Plastic Pipe Institute, USA.
- Haldar, A. and Mahadevan, S. (2000), *Reliability Assessment Using Stochastic Finite Element Analysis, Chapter 3: Fundamentals of reliability analysis*, Wiley and Sons, Canada.
- Hasofer, A.M. and Lind, N.C. (1974), "An exact and invariant first order reliability format", *Journal of Engineering Mechanics Division, ASCE*, **100**(12), 111-121.
- Hauch, S. and Bai, Y. (1999), "Bending moment capacity of pipes", *Proc. of the 18th International Conference on Offshore Mechanics and Arctic Engineering*, Newfoundland, Canada, July.
- Lee, O.S. and Kim, D.H. (2006), "Reliability of buried pipelines with corrosion defects under varying boundary conditions", *Journal of Solid State Phenomena*, **110**, 183-192.
- O'Reilly, M.P., Rosbrook, R.B., Cox, F.C. and McCloskey, A. (1989), "Analysis of defects in 180km of pipe sewers in southern water authority", *TRRL Research report* 172.
- Rackwitz, R. and Fiessler, B. (1978), "Structural reliability under combined random load sequences", *Computers & Structures*, **9**(5), 489-494.
- Rajani, B. and Makar, J. (2000), "A methodology to estimate remaining service life of grey cast iron water mains", *Journal of Civil Eng. National Research Council of Canada*, **27**, 1259-1272.

- Sarplast: IniziativeIndustriali S.P.A. (2008), "Installation Manual", 19-23, Santa Luce, Italy.
- Sadiq, R., Rajani, B. and Kleiner, Y. (2004), "Probabilistic risk analysis of corrosion associated failures in cast iron water mains", *Reliability Engineering and System Safety*, **86**(1), 1-10.
- Tee, K.F. and Li, C.Q. (2011), "A numerical study of maintenance strategy for concrete structures in marine environment", *Proc. of the 11th International Conference on Applications of Statistics and Probability in Civil Engineering*, Zurich, Switzerland, August.
- Tee, K.F., Li, C.Q. and Mahmoodian, M. (2011), "Prediction of time-variant probability of failure for concrete sewer pipes", *Proc. of the 12th International Conference on Durability of Building Materials and Components*, Porto, Portugal.
- Watkins, R.K. and Anderson, L.R. (2000), *Structural Mechanics of buried pipes*, CRC Press, LLC, Washington, D.C.USA.