Failure Analysis of Underground Pipeline Subjected to Corrosion

A thesis submitted in fulfilment of the requirements for the degree of Doctor of Philosophy

Weigang Wang

B. Eng (China University of Petroleum)

School of Engineering

College of Science, Engineering and Health

RMIT University

July 2018
Abstract

Underground pipes are essential infrastructure for the transport of water, oil and gas. The presence of localised pitting corrosion has been identified as one of the main deterioration mechanisms for metal pipes. When exposed to external loadings, these corroded pipes can easily fail due to intensified stresses at the corrosion pit. Disruptions to pipelines not only greatly affect the life of citizens, but also cause severe economic loss and pose safety risk. Therefore, accurate prediction of safe design life of buried pipes is significant. The main objective of this research is to investigate the effect of corrosion on the mechanical properties of cast iron pipes. A relatively long-term corrosion test was conducted on cast iron pipe in a corrosive clay soil. The corrosion behaviour of pipes was thoroughly examined using various corrosion techniques. At designated points of time, fracture toughness tests were conducted on single-edge bend specimens that were cut from the pipe wall. The results showed effective outcomes for corrosion behaviour in buried pipes and mechanical properties deterioration. A new three-dimensional geometrical model for sharp corrosion pits is proposed. The domain integral method has been employed, in conjunction with a three-dimensional finite element analysis, to derive the stress intensity factors for pipes. An expression of the maximum stress intensity factors has been developed for corroded pipes and the upcrossing method is employed to quantify the probability of fracture failure. This thesis concludes that both the mechanical properties and microstructure of material are changed due to corrosion. The proposed stochastic model of stress intensity factor can serve as a useful tool to predict the failure of buried cast iron pipes with improved accuracy. This research work will enhance the current knowledge of corrosion and mechanical property degradation of metal pipes and improve estimations of the remaining safe life of buried pipelines.
Declaration

I certify that except where due acknowledgement has been made, the work is that of the author alone; the work has not been submitted previously, in whole or in part, to qualify for any other academic award; the content of the thesis is the result of work that has been carried out since the official commencement date of the approved research program; any editorial work, paid or unpaid, carried out by a third party is acknowledged; and ethics procedures and guidelines have been followed.

Weigang Wang

17th July 2018
Acknowledgements

I would express my sincere gratitude and admiration to my senior supervisor, Professor Chun-Qing Li, for his guidance, support and encouragement that were unconditionally provided during my PhD studies. I learned a lot from his critical thinking and invaluable knowledge. I am extremely fortunate that he is my supervisor. Thank you very much.

I would also present great thanks to my second supervisors, Dr Annan Zhou and Dr Dilan Robert, for their continuous assistance and helpful advice throughout my research. It was my honour to work under their supervision and this extraordinary experience will benefit my future.

Grateful thanks are also extended to the technical staff in the School of Engineering and RMIT’s RMMF microscopy facility. These people include Pavel Ryjkov, Peg Gee Chang, Xiang Gao, Kevin le, Shamir Bhuiyan, Peter Tkatchyk, Peter Rummel, Edwin Mayes, Matthew Field, Andre Clemann and others. Without their kind help and patience, I would not have been able to finish my laboratory work. Special thanks also go to my colleagues and friends who brought me laughter and joy throughout the journey.

Finally, I owe a debt of gratitude to my parents and two elder sisters for the many sacrifices they made for me. The love gained from them is the light that shows me the way and gives me confidence and strength. I also wish to thank my partner, Wei Zhang, for her endless care and support in my life. Without her understanding, I would not have completed my thesis.
List of Publications

Journal Papers


Conference Papers


Contents

Abstract ......................................................................................................................... i
Declaration .................................................................................................................... ii
Acknowledgements ...................................................................................................... iii
List of Publications ....................................................................................................... iv
Contents ....................................................................................................................... vi
List of Figures ............................................................................................................... ix
List of Tables .............................................................................................................. xii
List of Abbreviations .................................................................................................. xiii

Chapter 1: Introduction ................................................................................................. 1
1.1 Introduction ............................................................................................................. 1
1.2 Significance of the Research ................................................................................ 3
1.3 Aims and Objectives ............................................................................................ 6
1.4 Scope and Dissertation Layout ............................................................................ 6

Chapter 2: Literature Review ....................................................................................... 8
2.1 Introduction ............................................................................................................. 8
2.2 Basics of Corrosion Science ................................................................................. 8
  2.2.1 General Corrosion .......................................................................................... 9
  2.2.2 Pitting Corrosion ......................................................................................... 10
  2.2.3 Corrosion Evaluation .................................................................................. 11
2.3 Corrosion of Pipe in Soils .................................................................................... 14
  2.3.1 Basics of Corrosion in Buried Pipes ............................................................. 15
  2.3.2 Formation of Corrosion Cells in Buried Pipes ............................................. 16
  2.3.3 Corrosion Tests of Buried Pipes .................................................................. 18
2.4 Factors Affecting Corrosion of Buried Pipes ..................................................... 25
  2.4.1 Water Content ............................................................................................ 25
  2.4.2 Soil Resistivity ............................................................................................ 26
  2.4.3 Soil pH .......................................................................................................... 27
  2.4.4 Soil Texture .................................................................................................. 28
  2.4.5 Pipe Material .............................................................................................. 28
  2.4.6 Environmental Factors ............................................................................... 29
2.5 Effect of Corrosion on Metal Pipes .................................................................... 30
  2.5.1 Effect of Corrosion on Pipe Strength .......................................................... 30
  2.5.2 Effect of Corrosion on Material Change ..................................................... 31
  2.5.3 Experiments on Pipes ............................................................................... 32
2.6 Basics of Fracture Mechanics ............................................................................ 37
  2.6.1 Stress Intensity Factor ............................................................................... 37
  2.6.2 Griffith’s Energy Theory ............................................................................. 39
  2.6.3 J-Contour Integral ...................................................................................... 39
  2.6.4 Methods of Determining Stress Intensity Factor ........................................ 40
2.7 Assessment of Buried Pipelines ........................................................................... 43
  2.7.1 Pipe Failure Models .................................................................................... 44
  2.7.2 Reliability Methods .................................................................................... 51
2.8 Summary ................................................................................................................ 56
# List of Figures

| Figure 1-1 | Examples of pipe failures in (a) Wednesbury, United Kingdom (Burst Wednesbury, 2018) and (b) Hobart, Australia (Hudspeth, 2018) | 2 |
| Figure 1-2 | Corrosion rates of pipes | 4 |
| Figure 2-1 | Potential-pH (Pourbaix diagram) for iron (Fe) | 9 |
| Figure 2-2 | Schematic of general corrosion process (adapted from Yang, 2010) | 10 |
| Figure 2-3 | Development of a pit in a solution saturated with sodium chloride (adapted from Davis, 2000) | 10 |
| Figure 2-4 | Standard rating chart for pits | 13 |
| Figure 2-5 | Schematic of corrosion in buried iron (adapted from Romanoff, 1957) | 15 |
| Figure 2-6 | Schematic of corrosion rate in buried iron | 16 |
| Figure 2-7 | Schematic of buried pipes (a) with two different soils contacted, (b) with water table lying between the bottom and top of pipe (c) in a trench with loose backfill | 17 |
| Figure 2-8 | Corrosion test setup in aerated tap solution | 20 |
| Figure 2-9 | Images of the arrangement of corrosion test specimens in (a) National Bureau of Standard’s data (Romanoff, 1957), (b) Norin and Vinka’s (2003) test and (c) specimen arrangement in Moore and Hallmark’s (1987) work | 22 |
| Figure 2-10 | Schematic of the test setup in a PVC tube corrosion cell | 24 |
| Figure 2-11 | Schematic diagram showing the current versus time curve of the Pt/Fe cell in soils with (a) water content = 10%, (b) water content > 10% and (c) saturated with water (Kreysa & Schütze, 2008) | 26 |
| Figure 2-12 | Schematic of intercrystalline corrosion in stainless steel | 33 |
| Figure 2-13 | Reduction of tensile strength with respect to corrosion-induced degradation | 34 |
| Figure 2-14 | (a) Typical testing setup for crush ring test (Seica & Packer, 2004) and (b) test apparatus of pipe bursting test (Netto, Ferraz & Estefen, 2005) | 35 |
| Figure 2-15 | Stresses near a crack tip | 38 |
| Figure 2-16 | Three basic modes of crack displacements | 39 |
| Figure 2-17 | Contour passes through Gauss points in a two-dimensional cracked body | 42 |
| Figure 2-18 | Typical setup of a three-point bending test | 43 |
| Figure 2-19 | Schematic of Schlick’s failure criterion (modified from Robert et al., 2016) | 46 |
| Figure 2-20 | Failure assessment diagram | 50 |
| Figure 2-21 | Geometric representation of the probability of structural failure | 52 |
| Figure 3-1 | Environmental chamber | 62 |
| Figure 3-2 | Schematic for the tests and specimen arrangement | 63 |
Figure 3-3 Fracture toughness specimens cut from pipe section ........................................ 66
Figure 3-4 Typical soil–pipe assembly and corrosion test setup ........................................ 67
Figure 3-5 Test setup for (a) fracture toughness and (b) modulus of rupture ...................... 70
Figure 3-6 Corrosion current density of pipes buried in soils with (a) pH 2.5, (b) pH 3.5 and (c) pH 5.0 .................................................................................................................. 72
Figure 3-7 Photos of corroded pipe specimens exposed to soil with various pH and exposure periods .............................................................................................................. 73
Figure 3-8 Corrosion potentials of pipes buried in soils with various pH ............................ 74
Figure 3-9 Corrosion rates of pipes in soil with various pH .............................................. 75
Figure 3-10 Reduction of fracture toughness in soil with various pH ............................... 77
Figure 3-11 Reduction of modulus of rupture in soil with pH 2.5 ..................................... 78
Figure 3-12 Measurements of pH in different soils .............................................................. 79
Figure 3-13 Typical X-ray diffraction results for (a) pH 2.5, (b) pH 3.5 and (c) pH 5.0 .. 81
Figure 3-14 Typical Raman spectra of rust formed on pipes buried in soil for (a) pH 2.5, (b) pH 3.5 and (c) pH 5.0 .................................................................................................. 83
Figure 3-15 Comparison of specimens after corrosion product removed with exhumed old pipes .......................................................... 84
Figure 3-16 Reduction of fracture toughness with mass loss ............................................ 86
Figure 3-17 Scanning electron microscope image cross-section of (a) pipe specimens without corrosion, (b) pipe specimens exposed to soil of pH 2.5 for 365 days (c) pipe specimens exposed to soil of pH 3.5 for 365 days and (d) a 37-year-old grey cast iron pipe ................................................. 88
Figure 4-1 Typical geometrical models for corrosion pits .................................................. 92
Figure 4-2 Typical configurations of corrosion pits (a) adapted from ASTM G46 (ASTM International, 2013a) and (b) observed in corrosion tests ................................. 93
Figure 4-3 Schematic for the generation of a sharp corrosion pit ................................. 95
Figure 4-4 A pressurised pipe with an external sharp corrosion pit .............................. 95
Figure 4-5 Domains of elements at the front of the sharp corrosion pit ......................... 99
Figure 4-6 Finite element model of a quarter of the pipe with a longitudinal sharp corrosion pit .......................................................... 100
Figure 4-7 Comparison of stress intensity factors along the crack front for (a) 
\[a/c = 1.0, \; d/R = 0.1 \; \text{and} \; b = 0, \; \text{and} \; (b) \; a/c = 2.0, \; \; d/R = 0.1 \; \text{and} \; b = 0........... 101
\]
Figure 4-8 Influence coefficients along pit front for (a) \(a/c = 0.4, \; d/R = 0.1\) and \(a/d = 0.5; \; (b) \; a/c = 0.8, \; d/R = 0.1\) and \(a/d = 0.5; \; (c) \; a/c = 2.0, \; d/R = 0.1\) and \(a/d = 0.5\) ......................................................................................................................... 104
Figure 4-9 Effect of pit width on maximum influence coefficient for (a) \(a/c = 0.4\) and \(d/R = 0.2\) and (b) \(a/c = 1.6\) and \(d/R = 0.2\) ............................................................................. 106
Figure 4-10 Comparison of results predicted by the evolutionary polynomial regression method and target values for (a) high aspect ratio case and (b) low aspect ratio case ........................................................................................................... 112
Figure 4-11 Comparison of maximum stress intensity factors, calculated from the derived formulas and literature .......................................................... 114
Figure 4-12 Comparison of maximum stress intensity factors calculated from finite element (FE) models and the derived formulas ......................... 115
Figure 5-1 Flowchart of the proposed methodology .......................................................... 119
Figure 5-2 Coefficient of determination ($r^2$) obtained from 73 locations ......................... 130
Figure 5-3 Boxplot for (a) $k$ in different groups of soils and (b) $n$ in different groups of soils ........................................................................... 136
Figure 5-4 Relationship between soil resistivity and total salt content ................................ 138
Figure 5-5 Relationships between (a) moisture equivalent and air-pore space, (b) volume shrinkage and moisture equivalent and (c) air-pore space and apparent specific gravity ..................................................................... 140
Figure 6-1 Probability-probability plot for $k$ and $n$ factors .................................................. 146
Figure 6-2 Comparison of experimental data and models predicted results for (a) $t = 1$ year, (b) $t = 5$ years, (c) $t = 10$ years and (d) $t = 30$ years ........................................ 149
Figure 6-3 A pressurised pipe with an external sharp corrosion pit .................................... 156
Figure 6-4 Distributions of the predicted corrosion pit depths by Equation 6.3 ................. 157
Figure 6-5 Flow chart of the Monte Carlo simulation .......................................................... 158
Figure 6-6 Probability of pipe fracture failure for different coefficients of correlation ($p$)........................................................................................................ 159
Figure 6-7 Probability of pipe fracture failure for different fracture toughness values ... 159
Figure 6-8 Change of probability sensitivity index with time .............................................. 160
List of Tables

Table 2-1 Values of $K$ for various units of corrosion rate ........................................... 12
Table 2-2 Relationship between soil resistivity and corrosivity ..................................... 27
Table 2-3 Summary of the effect of elements on mechanical properties of iron alloys .... 32
Table 2-4 Summary of common material failure criteria for metal .................................. 45
Table 2-5 Summary of the failure pressure of steel pipes subjected to corrosion defects ................................................... 49
Table 3-1 Physical properties of soil sample .................................................................. 60
Table 3-2 Chemical composition of soil sample (mg/kg) .................................................. 60
Table 3-3 Test variables in pipe section tests ................................................................. 64
Table 3-4 Chemical composition of cast iron ................................................................. 64
Table 3-5 Summary of the results of the fracture toughness test .................................. 76
Table 4-1 Variables in parametric finite element studies .............................................. 102
Table 4-2 Maximum influence coefficients for pipes with $d/R = 0.05$ ...................... 107
Table 4-3 Maximum influence coefficients for pipes with $d/R = 0.1$ ......................... 108
Table 4-4 Maximum influence coefficients for pipes with $d/R = 0.2$ ......................... 109
Table 4-5 Constants in Equation 4.10 and the fitting indicators ................................. 111
Table 5-1 Sample of corrosion measurements of cast iron pipes from the National Bureau of Standards database ................................................................. 121
Table 5-2 Sample of soil property information from National Bureau of Standards database .................................................................................................. 125
Table 5-3 Classification of soil aeration based on internal drainage tests from Soil Survey Manual .................................................................................................. 126
Table 5-4 Mean and coefficient of variance of $n$ and $k$ in different soil groups .......... 130
Table 5-5 Analysis of variance (ANOVA) results of $n$ and $k$ in different soil groups ... 131
Table 5-6 Correlation coefficient ($r$) between corrosion-influencing factors and $k$ and $n$ factors for different soil groups ...................................................... 132
Table 5-7 Correlation between soil resistivity and salt content .................................... 134
Table 5-8 Correlation between soil physical properties .................................................. 135
Table 6-1 Constants in influence coefficient functions (Equation 6.2) ....................... 144
Table 6-2 Statistics of distribution fitting for $k$ factor ................................................... 145
Table 6-3 Statistics of distribution fitting for $n$ factor ................................................... 145
Table 6-4 Values of basic variables ............................................................................... 157
### List of Abbreviations

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>3P</td>
<td>three parameters</td>
</tr>
<tr>
<td>ANOVA</td>
<td>analysis of variance</td>
</tr>
<tr>
<td>AVG</td>
<td>average error</td>
</tr>
<tr>
<td>CMOD</td>
<td>crack mouth opening displacement</td>
</tr>
<tr>
<td>CoD</td>
<td>coefficient of determination</td>
</tr>
<tr>
<td>COV</td>
<td>coefficient of variance</td>
</tr>
<tr>
<td>DC</td>
<td>direct current</td>
</tr>
<tr>
<td>EIS</td>
<td>electrochemical impedance spectroscopy</td>
</tr>
<tr>
<td>EPR</td>
<td>evolutionary polynomial regression</td>
</tr>
<tr>
<td>ER</td>
<td>electrical resistance</td>
</tr>
<tr>
<td>FAD</td>
<td>failure assessment diagram</td>
</tr>
<tr>
<td>FE</td>
<td>finite element</td>
</tr>
<tr>
<td>FORM</td>
<td>first-order second-moment</td>
</tr>
<tr>
<td>GEV</td>
<td>generalised extreme value</td>
</tr>
<tr>
<td>LPR</td>
<td>linear polarisation resistance</td>
</tr>
<tr>
<td>MIC</td>
<td>microbiologically-influenced corrosion</td>
</tr>
<tr>
<td>NBS</td>
<td>National Bureau of Standards</td>
</tr>
<tr>
<td>SCS</td>
<td>Soil Conservation Service</td>
</tr>
<tr>
<td>SEM</td>
<td>scanning electron microscope</td>
</tr>
<tr>
<td>SIF</td>
<td>stress intensity factors</td>
</tr>
</tbody>
</table>

xiii
<table>
<thead>
<tr>
<th>Acronym</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>SRB</td>
<td>sulphate-reducing bacteria</td>
</tr>
<tr>
<td>SSE</td>
<td>sum of squares of residuals</td>
</tr>
<tr>
<td>SST</td>
<td>total sum of squares</td>
</tr>
<tr>
<td>UK</td>
<td>United Kingdom</td>
</tr>
<tr>
<td>US</td>
<td>United States</td>
</tr>
<tr>
<td>XRD</td>
<td>X-ray diffraction</td>
</tr>
</tbody>
</table>
Chapter 1: Introduction

1.1 Introduction

Metal pipes are essential infrastructure for the transport of water, gas and oil. In Australia, there are approximately 210,000 km of buried pipes; of this, more than 70% are ferrous pipes, including cast iron, ductile iron and steel pipes (Cole & Marney, 2012). Cast iron pipes have had long-term use in most developed countries and currently, thousands of kilometres of cast iron pipes remain in networks and play a critical role in water and gas systems. However, due to increases in-service times and exposure to the corrosive environment, these pipes deteriorate and consequently, there has been an increasing number of pipe failures (Hou et al., 2016).

It is estimated that approximately 75% of the current pipe network is older than 25 years (Thomson & Wang, 2009). A recent report by Folkman (2018) showed that the overall break rates of water mains in North America increased by 27% from 11.0 to 14.0 breaks per 100 miles (approximately 160 km) per year, during the past six years. The report also showed that 82% of cast iron pipes in the United States (US) and Canada are older than 50 years and there has been a 46% increase in breaks in cast iron pipes since 2012 (Folkman, 2018). It was estimated that approximately 0.6% water mains are replaced each year in the US due to failure events (Cole & Marney, 2012). In Australia, over 20 pipe breaks per 100 km were reported per year. Consequently, Australia’s Urban Water Sector paid over A$90 million to replace the pipes (National Water Commission Australia, 2010). It was reported that between 2011 and 2012, there was an average 27% increase in the number of breaks per 100 km (National Water Commission Australia, 2013).

The failures of pipelines often occur unexpectedly, and the actual service life of a pipeline is significantly less than the design life expectancy. Recently, an underground water pipe burst in Wednesbury, United Kingdom (UK) on 26 April 2018 and the street was deluged (Burst Wednesbury, 2018). There was also an incident of a burst water main in Hobart, Australia on 15 March 2018, which caused the flooding of Hobart’s highway and resulted in an estimated total water loss of three million litres (Hudspeth, 2018). Therefore, the failure of pipelines not only
imposes an adverse impact on the integrity and performance of the pipe network, it also causes significant economic loss to water utilities and the public in general (see Figure 1-1).

Figure 1-1 Examples of pipe failures in (a) Wednesbury, United Kingdom (Burst Wednesbury, 2018) and (b) Hobart, Australia (Hudspeth, 2018)

It is widely recognised that corrosion is the main cause of material degradation and structural deterioration. A survey by the National Research Council of Canada reported that 23 out of 25 mechanical failures investigated were associated with corrosion (Boudreau & Brynildsen, 2003). The most common failure modes of cast iron pipes include through hole, blowout, longitudinal crack and circumferential crack (Makar et al., 2001). In general, the presence of corrosion in a pipe not only causes pipe wall thinning, it can also induce stress redistribution and concentration around the corrosion area. Compared with uniform corrosion, pitting corrosion is more detrimental to pipe integrity because the growth of corrosion pits can result in leakage and failure with a small amount of material loss.
Soil was reported to be responsible for approximately 65% of external corrosion in buried iron, while stray currents and contact with other metals account for 35% (Kreysa & Schütze, 2008; Romer & Bell, 2001). Due to the existence of mineral salts and water content, soil is generally assumed to be a good electrolyte for corrosion (Pritchard et al., 2013). It is widely recognised that soils with poor aeration, high electrical conductivity and moisture content and high levels of soluble salts (e.g., clay soils) are corrosive soils (Cole & Marney, 2012; Kreysa & Schütze, 2008; Romanoff, 1964). A recent report by Folkman (2018) demonstrated that the number of breaks of cast iron pipes in corrosive soils is 20 times more than pipes in less corrosive soils. It was even reported that some cast iron pipes in the San Diego (California, US) water network system experienced perforation within two years of installation (Davis, 2000). With consideration of the fact that approximately 70% of the total assets in worldwide water utilities consist of buried pipes (Petersen & Melchers, 2012), it is necessary to thoroughly investigate the corrosion behaviour of metal pipes in corrosive clay soils and determine the effect of corrosion on the mechanical properties of pipes. This will enable the accurate prediction of pipe failures and service life.

1.2 Significance of the Research

The unprecedented high frequency of breaks of in-service pipelines draw an increasing attention from industries and users. There are completed and ongoing research programs funded by industrial and academic bodies (e.g., Water Research Foundation, Melbourne Water and UK Water Industry Research), which are aimed to predict the failures of pipes. However, the occurrences of pipe failure events have not been effectively predicted and prevented, mainly due to the insufficiency of research into material deterioration and failure mechanisms and the lack of advancement in assessment methods. This motivates current research programs to advance the theory of pipe failures and develop accurate assessment methods.

The corrosion rate of a buried pipe is the result of the functions of many soil properties, not one single parameter. Although it is well-known that the corrosion rate of buried pipes is initially high and decreases gradually with the increase of exposure time (as shown in Figure 1-2), the determination of corrosion rates in specific soils is complicated by various chemical agents, aeration and the
heterogeneity of soils. There is considerable amount of research (e.g., Gupta & Gupta 1979; Mohebbi & Li, 2011; Moore & Hallmark, 1987; Murray & Moran, 1989; Norin & Vinka, 2003; Wu et al., 2010) that has investigated the corrosion behaviour of buried pipes. Most of these studies are based on solutions or simulated soil solutions tests (e.g., Liu et al., 2010; Wu et al., 2010), or based on small-scale specimens that have been subjected to a short-time exposure period (i.e., in days or weeks; e.g., Gupta & Gupta, 1979). The validity of applying the data derived from these tests to predictions about pipe corrosion behaviour in real soils is debated. For example, it is argued (Ferreira et al., 2007) that the electrochemical parameters that were derived from soil solution tests were observed in field observations. As corrosion data collected from the field is often subjected to different issues (e.g., lack of soil properties information and uncertainties concerning pipe materials and size), a weak correlation between soil parameters and corrosion rate is observed (e.g., Doyle et al., 2003; Norin & Vinka, 2003). Therefore, it is necessary to conduct corrosion tests to replicate the corrosion of pipes in the real soil to understand corrosion behaviour in buried pipes and the influencing factors.

![Figure 1-2 Corrosion rates of pipes](source: Li and Mahmoodian (2013)).

The effects of corrosion on the structural integrity of pipelines have been intensively explored. Most of the previous studies were based on strength theory (e.g., American Petroleum Institute, 1999; Ahammed, 1998; Kiefner & Vieth, 1989), such as flexural failure and rupture. Few studies were based on fracture mechanics (e.g., Conlin & Baker, 1991; Hou et al., 2016). Fracture failure is a relatively common occurrence,
especially for cast iron pipes, due to the brittle nature of cast iron material. Fracture-related failure is expensive to repair and, if they occur in gas mains, can lead to explosions (Conlin & Baker, 1991; Makar et al., 2001). Despite the practical significance of this observation, little research has been conducted to investigate the effects of corrosion pits on pipe failures, based on fracture mechanics. It is also known that fractures are related to sharp crack defects or notches (Conlin & Baker, 1991; Fry & Rumsey, 1983). Cast iron is characterised by the presence of graphite flakes in the matrix of iron. The graphite flakes can act as micro crack initiators within corrosion pits. As a result, sharp tips often form on the surface of corrosion pits (Conlin & Baker, 1991). Subsequently, the collapse of pipes occurs when the fracture toughness of pipe is exceeded. A review of the literature indicates that most of the previous research was based on assumption that corrosion has blunt pit geometries, which include the rectangular shape, semi-ellipsoid or circular cone. Little research has focused on the effect of sharp corrosion pits on pipes, with exception of few works by Li and Yang (2012), Raju and Newman (1982) and Atluri and Kathiresan (1980). However, the flaws considered in these studies were semielliptical surface cracks, rather than three-dimensional cavities (corrosion defects) with a sharp front. Therefore, it is necessary to investigate the effect of sharp corrosion pits on pipes.

Many parameters (e.g., corrosion rates, material properties and loading conditions) in a practical pipe system are uncertain and time-variant during its service life. To consider the uncertainty and time-variant of these parameters, it is reasonable to represent one parameter (e.g., corrosion depth) or a combination of several parameters (e.g., the loading effect) by a stochastic process. To cope with the problem of a stochastic process against a limit (i.e., threshold), it is necessary to incorporate an upcrossing theory-based method into the failure assessment of corroded pipes. In contrast to traditional deterministic methods, an assessment method considering the time-variant characteristics of parameters in a pipe system will greatly increase the accuracy of failure prediction.

In summary, a comprehensive review of buried pipe failure analysis shows that there are necessary areas for further investigation, to enrich current knowledge of pipe corrosion in corrosive soils, material deterioration and assessment of corrosion effect
on the integrity of pipes. This gives rise to the need for the present research, in which the important aspects of pipe failure are integrated into a methodology to develop a sustainable solution for failure analysis of the vast asset of buried metal pipes in the world.

1.3 Aims and Objectives

The main aim of this research is to develop a new method that allows accurate prediction of the remaining safe life of metal pipes buried in soil. It integrates corrosion science, fracture mechanics and time-dependent reliability theory into a methodology to analyse corrosion behaviour of buried pipes, the degradation of material and mechanical properties, and the failure probability of corroded pipes. The specific objectives of this research are as follows:

- understand the fundamentals of corrosion science, fracture mechanics and pipe assessment methods
- investigate the corrosion behaviour of cast iron pipe buried in soils and understand the relationship between corrosion rate and exposure time
- investigate the effect of corrosion on pipe mechanical properties degradation over time and develop a model of fracture toughness deterioration
- investigate the effect of sharp corrosion pits on stress intensity factors (SIFs) of pipes and develop a relationship between the maximum SIF and the geometries of pits and pipe
- investigate the dependence of corrosion on soil properties and develop a probabilistic corrosion model
- develop a time-dependent reliability method, which incorporates the concepts of fracture mechanics, stochastic process and upcrossing.

1.4 Scope and Dissertation Layout

This thesis will focus on buried cast iron pipes subjected to corrosion.

This thesis consists of seven chapters.

Chapter 1 presents the background of the research, significance, aims and objectives, and the layout of the dissertation.
Chapter 2 is a literature review. In this chapter, basic theories, methods and relevant state-of-the-art research are critically reviewed, including the fundamentals of corrosion in buried pipes, factors influencing corrosion, effects of corrosion on pipes and pipe failure assessment methods.

Chapter 3 presents a comprehensive experimental program on corrosion behaviour and the corrosion effect on mechanical properties of buried cast iron pipes. A laboratory burial test is developed to replicate the corrosion of pipes in clay soil and the results of degradation of cast iron mechanical properties (fracture toughness and modulus of rupture) are presented.

Chapter 4 presents a finite element (FE) analysis of the SIF of cast iron pipes that are subjected to sharp corrosion pits. A three-dimensional geometrical model is proposed to represent sharp corrosion pits. An expression of the maximum SIF has been developed for corroded pipes.

Chapter 5 presents a comprehensive statistical analysis of factors affecting corrosion of buried cast iron pipes. A historical corrosion database is examined and the dependency of corrosion on soil properties is thoroughly investigated using soils with different aeration levels.

Chapter 6 presents a time-dependent reliability analysis of corroded pipes. A probabilistic corrosion model is developed; the probability of fracture failures is calculated by modelling the loading effect as a lognormal stochastic process.

Chapter 7 presents the conclusions of the thesis, along with the recommendations for future work.
Chapter 2: Literature Review

2.1 Introduction

To achieve the proposed aims and objectives, the fundamentals of corrosion in buried pipes, corrosion-influencing factors, effects of corrosion on pipes and pipe failure assessment methods are necessary. This chapter reviews important aspects that are required to analyse the failure of buried pipes due to corrosion and predict the service life of buried pipelines. As corrosion is the main factor that causes the deterioration of material and pipe structures, it is first reviewed. This is followed by the description and analysis of corrosion-influencing factors and the effect of corrosion on pipe material and mechanical properties. To analyse the fracture failures of pipes with sharp corrosion pits, the basics of fracture mechanics are discussed. Finally, failure assessment methods that are used to evaluate the remaining safe life of buried pipelines, subjected to time-variant loading and corrosion processes, are outlined.

2.2 Basics of Corrosion Science

The corrosion of iron is essentially an electrochemical process. It occurs when two or more points on a metal surface have a potential difference and two reactions, oxidation (i.e., anodic) and reduction (i.e., cathodic), occur simultaneously (Cramer & Covino, 2003). By definition, the anodic reaction of iron is the removal of electrons from the metallic state, resulting in a nonmetallic state and valence increase as follows (Marcus, 2003):

\[ \text{Fe} \rightarrow \text{Fe}^{2+} + 2e^- \]  \hspace{1cm} (Equation 2.1)

The balancing reaction of an anodic reaction is the reduction reaction, in which ions in the electrolyte accept electrons that are released from the electrically connected anode point, such as the liberation of hydrogen and the reduction of oxygen as follows (Cramer & Covino, 2003):

\[ 2\text{H}^+ + 2e^- \rightarrow \text{H}_2 \]  \hspace{1cm} (Equation 2.2)

\[ \text{O}_2 + 4\text{H}^+ + 4e^- \rightarrow 2\text{H}_2\text{O} \text{ (in acid solution)} \]  \hspace{1cm} (Equation 2.3)
The potential-pH (Pourbaix) diagram is frequently used to determine the stability of iron in a given corrosion environment. Figure 2-1 shows the potential-pH diagram of iron in water. The horizontal bottom line indicates a limit below which iron is stable and corrosion does not take place; however, iron dissolves into Fe$^{3+}$ or Fe$^{2+}$ in the zone above this limit. The potential-pH diagram is a useful tool that is often used for the interpretation of the stability of iron; however, it does not provide information on the rates of corrosion reactions. Further, as the potential-pH diagrams are often constructed through thermodynamic calculations based on few chemical species, the application of such a potential-pH diagram in a practical corrosion system with many more chemical species presented may cause a serious error.

![Potential-pH diagram for iron (Fe)](image)

**Figure 2-1 Potential-pH (Pourbaix diagram) for iron (Fe)**


### 2.2.1 General Corrosion

Corrosion can be broadly classified into two categories: general corrosion and pitting corrosion. As discussed in Section 2.2., a couple of anodic and cathodic reactions...
constitute a corrosion cell. If multiple micro local corrosion cells exist on the surface, uniform corrosion can occur (Davis, 2000). This is often called general corrosion. The general corrosion commonly induces a uniform thinning or penetration of the entire surface of an infrastructure (e.g., pipeline), as illustrated in Figure 2-2.

![Figure 2-2 Schematic of general corrosion process (adapted from Yang, 2010)](image)

2.2.2 Pitting Corrosion

Pitting corrosions have relatively small dimensions. They are either isolated on the surface, or as many pits occur close together, they resemble a roughened surface (Davis, 2000). Many alloys (e.g., iron, copper, aluminium and steels) are often subjected to pitting corrosion (Marcus, 2011). The rate of pitting corrosion is more rapid, due to the small anode area with respect to the large cathode area, resulting in the acceleration of metal dissolution (Cramer & Covino, 2003).

![Figure 2-3 Development of a pit in a solution saturated with sodium chloride (adapted from Davis, 2000)](image)

One of the common mechanisms of pitting is an autocatalytic reaction within a corrosion pit. This is often caused by the change of local environment (e.g., the
corrosion process of iron exposed to sodium chloride solution; see Figure 2-3). The dissolved oxygen near the local pit becomes depleted due to the progress of corrosion. This cause the cathodic area to move away from the pit and a spatial separation between the cathode and anode areas is formed. Subsequently, the iron ions produced from the cation hydrolysis of iron in the cavity of pit cannot be neutralised by the hydroxide ions produced in the cathodic reaction, causing the pH in the pit to decrease and the propagation of the pit continue (Frankel, 2003).

Materials with the presence of passive (oxide) film on the surface are susceptible to pitting corrosion. A more complex corrosion process, comprising of a sequence of steps, takes place (Marcus, 2011). In general, the initiation of pitting corrosion in passivated metals is often attributed to (1) diffusion of halides, (2) thinning of passive film and (3) mechanical ruptures (Marcus, 2011).

Pitting corrosion in metals can also be caused by other conditions. For example, the surface deposit, which creates local concentration cells of highly corrosive agents, contact a certain area of surface, causing rapid dissolution of metal in this area with respect to the rest of the surface (Davis, 2000).

### 2.2.3 Corrosion Evaluation

Generally, it is relatively simple to test and evaluate uniform corrosion. In laboratory corrosion tests, coupons are usually immersed in simulated solutions or exposed to actual service environments (Phull, 2003a). The guidelines for conducting immersion tests using coupons can be found in NACE International’s (2000) ‘TM0169’ and ASTM International and NACE International’s (2012) ‘G31’. The standards for cleaning corroded specimens and mass loss measurements can be found in ASTM International’s (2011) ‘G1’. Mass loss is calculated as a reduction of specimens before and after corrosion, often expressed in g/cm². The thickness reduction of the testing specimens can be calculated based on mass loss measurement or directly measured by micrometres or callipers (Phull, 2003a). If only uniform corrosion occurs, the average corrosion rate can be determined by mass loss measurement as shown in Equation 2.5 (ASTM International, 2011), in which $cr$ is corrosion rate (for units, see Table 2-1), $K$ is a constant (see Table 2-1), $T$ is corrosion exposure...
time (in hours), $A$ is specimen exposure area (in cm$^2$), $W$ is the mass loss (in grams) and $D$ is material density (g/cm$^3$):

$$cr = \left(\frac{K \times W}{A \times T \times D}\right)$$  

*(Equation 2.5)*

### Table 2-1 Values of $K$ for various units of corrosion rate

<table>
<thead>
<tr>
<th>Units of Corrosion Rate</th>
<th>$K$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mils per year (mpy)</td>
<td>$3.45 \times 10^6$</td>
</tr>
<tr>
<td>Inches per year (ipy)</td>
<td>$3.45 \times 10^3$</td>
</tr>
<tr>
<td>Micrometres per year ($\mu$m/y)</td>
<td>$8.76 \times 10^7$</td>
</tr>
<tr>
<td>Millimetres per year (mm/y)</td>
<td>$8.76 \times 10^4$</td>
</tr>
<tr>
<td>Grams per square metre per hour (g/m$^2$ h)</td>
<td>$1 \times 104 \times D$</td>
</tr>
<tr>
<td>Milligrams per square decimetre per day (mdd)</td>
<td>$2.4 \times 106 \times D$</td>
</tr>
</tbody>
</table>

*Source: ASTM International (2011).*

The rates of uniform corrosion can also be estimated by electrochemical methods, (e.g., linear polarisation resistance [LPR] and electrical resistance [ER]; Phull, 2003a). In the LPR method, polarisation resistance is experimentally determined and associated to the corrosion current density through the Stern-Geary equation as follows (Andrade & Alonso, 1996; ASTM International, 2004):

$$i_{\text{corr}} = \frac{B}{R_p}$$  

*(Equation 2.6)*

$$B = \frac{\beta_A \beta_C}{2.303 (\beta_A + \beta_C)}$$  

*(Equation 2.7).*

In the Stern-Geary equations (Equations 2.6 and 2.7), $i_{\text{corr}}$ is the corrosion current density (mA/cm$^2$), $R_p$ is polarisation resistance (ohm-cm$^2$) that can be determined by potentiodynamic polarisation resistance measurement (ASTM International, 2014) or stepwise potentiostatic polarisation measurement (ASTM International, 2004), $B$ is the Stern-Geary constant and the Tafel slopes $\beta_C$ and $\beta_A$ (mV/decade) are either experimentally measured or estimated based on experience (Andrade & Alonso, 1996; Millard et al., 2001). In ER measurement, the values of ER of particularly designed probes are monitored and the corrosion rate of the corroding system is directly calculated by determining the slope of the curve of resistance versus time.
(Phull, 2003a). A merit of the ER method is that a conductive medium is not required (Phull, 2003a). However, the accuracy of the ER measurement is highly dependent on temperature.

For pitting corrosion, several standards (see ASTM International, 2011, 2013a) are available to provide procedures for visual and metallographic examination of corrosion pits, pit distribution observation and dimension measurement. A micrometre is commonly used to measure the depths of pits. However, it is difficult to measure a narrow pit by use of a micrometre (Flitton & Escalante, 2003). Instead, an optical means (e.g., optical microscopy or scanning electron microscope [SEM]) can be used to perform cross-section measurements; however, it is a time-consuming and destructive method.

In a rough surface assessment, pitting factor—defined as the ratio of deepest corrosion depth to averaged corrosion depth—is often used to describe the extent of pits (Phull, 2003b). Another practical of way to characterise corrosion pits is to reference the standard chart, as shown in Figure 2-4 (ASTM International, 2013a).

![Figure 2-4 Standard rating chart for pits](source: ASTM International (2013a))
As the distribution and depths of pitting corrosion are random in nature, statistics have been widely applied to evaluate the extent of corrosion pits (e.g., Alamilla & Sosa, 2008; ASTM International, 2013a; Valor et al., 2010). For example, Scott (1934) proposed a method of estimating the deepest corrosion pit on the surface of a structure by use of a small portion of the area and completing the following steps:

1) measure the average maximum pit depth of unit areas in a corroded pipe sample
2) increase the unit areas twice and measure the averaged maximum pit depth
3) increase the areas successively and measure the averaged maximum pit depth until the entire surface area is covered
4) plot the logarithms of averaged maximum pit depth against the logarithms of areas in the x–y coordinate. A straight line is then obtained as \( p_m = bA^a \), in which \( p_m \) is the maximum pit depth in area \( A \) and \( b \) and \( a \) are constants depending on the metal material and exposure conditions.

In summary, corrosion is essentially an electrochemical process; however, general and pitting corrosion occur due to different formation mechanisms. The process of pitting corrosion is more complex, compared with that of general corrosion. Pitting corrosion often grows rapidly and thus, is more detrimental to civil infrastructures, particularly pipelines. The evaluation of uniform corrosion is relatively straightforward; however, it is difficult to obtain the rate of pitting corrosion in engineering corrosion environments. Understanding the fundamentals of corrosion reactions in metal is of importance to accurately predict the rates of pipe failures.

### 2.3 Corrosion of Pipe in Soils

Pipelines are generally designed with specified dimensions and materials to withstand the pressurised content (e.g., water, oil and gas) and external loadings, with some tolerances. However, with the increased service time requirement, there is an increasing number of pipe failures due to the deterioration of material. The exposure of pipes to some aggressive matter, such as corrosive soils, often worsens the deterioration. As such, there is an unexpected high rate of breaks happening in the current pipe network. Experience of pipe failures suggests that corrosion is the
main cause of deterioration in buried pipes. In Section 2.3, both the corrosion of pipes buried in soils and the previous studies on buried metal corrosion are reviewed.

2.3.1 Basics of Corrosion in Buried Pipes

The development of corrosion in buried pipes is a complex and time-variant process. Generally, a higher corrosion rate is expected once the corrosion process has begun. The corrosion rate decreases and becomes controlled by the diffusion of oxygen to the metal surface through the matrix of soil. As corrosion further progresses, a protective rust barrier normally forms between uncorroded metal and the surrounding soil. This rust layer can slow the transportation of reactants (H⁺ or O₂) to the material substrate, resulting in a decreased corrosion rate (Rossum, 1969). A schematic of a corrosion pit in buried iron is shown in Figure 2-5.

The corrosion rate of pipes is found to vary widely depending on its exposure conditions. According to Davis (2000), the corrosion rates of buried iron vary from 2 mm/year to 20 mm/year in different environments, according to their exposure to acidity and temperature. A biocorrosion model (see Figure 2-6), which was originally developed to characterise the corrosion growth in steel exposed to a marine environment, is applicable to the corrosion of buried ferrous pipes (Petersen &
Melchers, 2012; Melchers & Jeffrey, 2008). The general trend of corrosion growth presented by this model was found consistent with Romanoff’s data (Romanoff, 1957) in a relatively short-time exposure (less than 10 years) (Petersen & Melchers, 2012). This biocorrosion model shows that the corrosion of buried iron grows at a steady rate after 10 to 20 years of corrosion exposure (Petersen & Melchers, 2012).

![Figure 2-6 Schematic of corrosion rate in buried iron](image)

Source: Petersen and Melchers (2012).

### 2.3.2 Formation of Corrosion Cells in Buried Pipes

There are several different mechanisms that cause corrosion cell formations in pipes that are buried in soils. The most common corrosion phenomenon is caused by the variations of dissolved oxygen concentration at the pipe’s surface. In most cases, this is induced by the differences in oxygen and the moisture of soil in which the pipe is buried (Darbin et al., 1979; Romanoff, 1964). For example, when a pipe is buried in a soil mixture consisting of moist clay lump and sandy soil, which have different oxygen permeabilities, the surface area contacting clay lump (with low levels of dissolved oxygen) serves as the anodic area with pitting (see Figure 2-7a). Corrosion also occurs at the bottom area of a pipe, if the water table remains in the middle of buried pipe, as shown in Figure 2-7b. In general, the soil at the bottom of the pipes is more compact and badly aerated (e.g., when pipe is buried in a trench with loose backfill and natural undistributed soil at the bottom, as shown in Figure 2-7c). This causes an oxygen concentration cell between the top and bottom surface of the buried pipe, with the badly aerated area (i.e., the bottom) acting as the anode. Consequently, the bottom of buried pipe is often severely corroded.
Figure 2-7 Schematic of buried pipes (a) with two different soils contacted, (b) with water table lying between the bottom and top of pipe (c) in a trench with loose backfill

Source: Petersen and Melchers (2012).

The corrosion of buried pipes can also be caused by the current flow that is generated by electrically connecting two dissimilar materials (Thomson & Wang, 2009). This form of corrosion is known as galvanic corrosion. The cause of this type of corrosion is that one metal is more noble than the other and the active metal acts as the anode that loses electrons. For example, when a section of copper pipe is connected to a
ferrous pipeline, galvanic corrosion occurs with iron pipe sections serving as the anode (Thomson & Wang, 2009).

Stray current corrosion is a special electrolytic corrosion that occurs in buried metal pipes. This type of corrosion is induced when a stray electrical current that originates from a direct current source passes through metal pipes to find a high-conductivity path (Davis, 2000). Severe corrosion damages occur at the point from which the stray current leaves the earth and enters the pipe (Thomson & Wang, 2009). Compared with other forms of corrosion, stray current corrosion is independent of soil properties (Davis, 2000).

Microbiologically-influenced corrosion (MIC) is another type of corrosion that often causes the acceleration of material degradation in buried pipes (Beech & Sunner, 2004). This type of corrosion is caused by the presence and activities of microorganisms. Usher, Kaksonen, Cole and Marney (2014) presented a critical review of a MIC mechanism and the impact of specific microorganisms (e.g., sulphur-oxidising microorganisms, methanogens, fungi and sulphate-reducing bacteria [SRB]) on the external corrosion of buried steel pipes. Although many microorganisms can affect the processes of corrosion, the main culprits of corrosion in buried pipes are SRB (Sanders & Hamilton, 1985). According to Davis (2000), the corrosion rate of iron, with the presence of anaerobic SRB, can be 20 times greater than in a sterile condition. The impact of SRB is attributed to its capability to produce hydrogen sulphide (H₂S), which can intensely attack iron (Davis, 2000).

2.3.3 Corrosion Tests of Buried Pipes

There have been many tests conducted to investigate the corrosion behaviour of buried pipes in the past decades. These tests can be broadly categorised into three types: immersion tests, field burial tests and laboratory pipe-soil tests.

2.3.3.1 Immersion Tests

The immersion tests have been frequently used to study the corrosion mechanism and resistance of new pipe materials of interest (e.g., Belmokre et al., 1998; Benmoussa et al., 2006). Most of them were designed to investigate the effect of chemical compositions (Ca²⁺, K⁺, SO₄²⁻, HCO₃⁻ and NO₃⁻), pH, dissolved oxygen
and temperature on the corrosion behaviour of metals (Liu et al., 2010; Nie et al., 2009; Wu et al., 2010 etc.).

Specifically, Belmokre et al. (1998) investigated the corrosion behaviour of bare X60 steel plates and plates coated with primer in both soil-simulated solution and 3% NaCl solution. Benmoussa et al. (2006) conducted similar tests on X60 steel exposed to soil-simulated solution only, with temperatures ranging from 20 °C to 60 °C. The electrochemical impedance spectroscopy (EIS) measurements were used to estimate the corrosion rates of specimens. Liu et al. (2010) conducted corrosion tests of Q235 steel in simulated soil solutions, made with 0.01 M NaCl solution and content of chemical agents of a soil in Yingtan, China. The electrochemical methods used for corrosion rates estimation included potentiodynamic polarisation and EIS measurements. Liu et al. (2010) found that some cations and anions (e.g., K⁺, NO³⁻, SO₄²⁻) increased corrosion rates, while some others (e.g., Mg²⁺ and Ca²⁺) decreased corrosion rates. Hence, a ranking of chemical components in terms of their corrosion aggressiveness was established by Liu et al. (2010). Wu et al. (2010) investigated the influence of pH (i.e., 3.0, 4.0, 5.5 and 7.0) on corrosion behaviour of Q235 steel using a similar method to Liu et al. (2010). The results demonstrated that corrosion rates decrease with the increase of pH value. Liu et al., (2014) conducted corrosion tests of 13Cr steel in an autoclave with brine completion fluids (3MPa and 150 °C) for a maximum exposure time of 60 days. The pit evolution and growth were investigated with the aid of SEM. It was found that the specimen was susceptible to pitting corrosion and Br⁻ was attributed to the breakdown of the oxide film and growth of pitting corrosion.

As reviewed, there are many immersion tests conducted previously to investigate the resistance of steel alloys in environments of interest. However, relatively little research has been conducted on cast iron pipes, with the exception of a few recent works by Mohebbi and Li (2011) and Hou et al. (2016). Mohebbi and Li (2011) conducted immersion corrosion tests on cast iron plates that were cut from ex-service cast iron pipes (see Figure 2-8). The solutions used were prepared with various aeration and salt levels. This study revealed that the microstructure of materials plays a significant role in corrosion behaviour and localised corrosion is a main deterioration form in cast iron pipe materials. Hou et al. (2016) conducted similar
tests on HT 200 cast iron and Q235 steel in soil-simulated solutions under three different pH levels (3.0, 5.5 and 8.0). Various measurement techniques, including SEM, X-ray photon spectroscopy, X-ray diffraction (XRD), DC polarisation and electrochemical frequency modulation, were used to examine the corrosion-induced damage and corrosion rates during the tests. These two studies are scientifically significant because of the relatively long corrosion time used (i.e., 360 days in Mohebbi & Li, 2011; 270 days in Hou et al., 2016).

Figure 2-8 Corrosion test setup in aerated tap solution

Source: Mohebbi and Li (2011).

It can be observed that immersion tests are easy to create and the environmental conditions (e.g., temperature, pH and salt content) are easy to control. Small specimens (e.g., coupons) are usually exposed to a small volume of soil-simulated solution that is prepared either by extracting the chemical agents from soils or based on the information in literature. Most of tests were short term and conducted in hours or days (e.g., Liu et al., 2010; Nie et al., 2009; Sancy et al., 2010; Wu et al., 2010). Further, most of the immersion tests were accelerated by either increasing the severity of the environmental exposure (e.g., more acidic or higher temperatures) or impressing the external current. However, the significance of soil solid phase is often not considered. It has been found that electrochemical parameters, derived from soil solution tests, were often observed differently when compared to field corrosiveness tests (Ferreira et al., 2007). The relevance of soil-simulated solution tests to replicate the real service soil condition remains uncertain (Cole & Marney, 2012).
2.3.3.2 Field Burial Tests

Many field buried tests have been conducted in the past to assess pipe corrosion (e.g., Moore & Hallmark, 1987; Norin & Vinka, 2003; Romanoff, 1957). In field buried corrosion tests, metal specimens are placed in actual service environments to evaluate the corrosion resistance of materials. A commonly used technique is to bury metal specimens in a ditch with the approximate depth of 1.2 m as shown in Figure 2-9a (Cramer & Covino, 2003; Romanoff, 1957). Typical specimens are coupons, metal plates, pipe sections or pipe fixtures that are placed approximately 300 mm apart (Cramer & Covino, 2003). Corrosion rates are often determined by measuring the mass loss or thickness reduction after a known or prescribed exposure time (Phull, 2003a).

For example, the US National Bureau of Standards (NBS; Romanoff, 1957) conducted comprehensive field burial corrosion tests to investigate the performance of commonly used metals and alloys in various locations throughout the US. More than 30,000 metal specimens, including nuts, bolts and pipe sections, were buried in 128 different soils with the corrosion exposure of up to 19 years (see Figure 2-9a). The duplicates of specimens were examined approximately every two years for mass loss and maximum pit depth measurements. Results showed that corrosion rates of pipes greatly vary with respect to soil properties (e.g., texture, aeration, salt content and resistivity) and less to metal materials.

Moore and Hallmark (1987) investigated the corrosivity of soils in 16 locations in Texas, US by using mild steel specimens (American wire gauge with 1.6 mm diameter) for a period of one year at three different time points (i.e., 6, 9 and 12 months). Two burial depths (i.e., 0.6 mm and 1.5 mm) were considered. The corrosion-induced material damage is presented as the reduction of wire cross area, which is calculated from the increased electrical resistivity of specimens. Results indicated that the original SCS system for soil corrosivity classification, based on the NBS corrosion database, is too conservative. A revised criterion was developed, based on soil particle size, resistivity, extractable acidity and wetness class. Although the tests of Moore and Hallmark (1987) were significant, a high variance of data was observed, due to temperature fluctuations and variabilities in spatial and temporal soil properties.
Norin and Vinka (2003) conducted a three-year field burial test with steel panels buried in different depths (i.e., 0.5 m, 1.1 m and 1.5 m) in soils of an urban environment. Different measurements, including mass loss, half-cell potential measurement, ER and LPR, were used. The pitting corrosion rates were found to be greater in filling material than in other soils. The physical properties of soil were found to have a greater impact on pitting corrosion than the chemical agents in soils. It was also found that the corrosion rates of specimens were greatly influenced by the seasons.

Field burial tests most closely replicate real service conditions and reliable data are expected to be produced for use in engineering practices. However, the data produced from field burial tests are often subject to a high degree of variation due to the seasonable variation of environments (Norin & Vinka, 2003). Most of previous studies (e.g., Moore & Hallmark, 1987; Norin & Vinka, 2003; Romanoff, 1957, 1964; Sancy et al., 2010) lack detailed information about soil properties, specifically moisture content, pH, air-pore space, temperature and microbes. This significantly limits the analysis of corrosion data obtained from field burial tests and their
engineering applications. Moreover, field burial tests often require significant time and have high costs and therefore, are not widely used.

2.3.3.3 Laboratory Pipe-Soil Tests

Alternatively, corrosion tests in soil can be conducted in a controlled laboratory environment. The results produced from laboratory pipe-soil tests correlated well with corrosion in the field, if properly designed and operated. For example, Schwerdtfeger (1953) conducted a series of corrosion tests on cast iron and steel specimens using 10 different soils selected from the NBS’s fieldwork. The results produced from the laboratory tests were compared with the results obtained from the field tests. A good correlation was obtained in nine (out of 10) soils. Some significant results of corrosion in buried iron are observed in laboratory pipe-soil tests, in which environment conditions are carefully monitored. For example, Goodman et al., (2013) found that the corrosion rate of specimens buried at 100 mm varies greatly with respect to moisture variations, while the corrosion rate of specimens buried at 400 mm varies with respect to the availability of oxygen. Gupta and Gupta (1979) found that the corrosion rates of specimen reached a maximum when the moisture content was 65% of its water-holding capacity. This critical moisture content is generally 25–35% of water content by weight (Gupta & Gupta, 1979). The test results of Murray and Moran (1989) further implied that the critical moisture content, in terms of the corrosion aggressiveness of soil, depended on the texture of soils and other soil properties. More recently, Nie et al. (2009) found that in temperatures over 50 °C, the passivation of steel material no longer remained and material started to actively dissolve. The laboratory pipe-soil tests of López et al., (2006) showed that induction currents that exist in nature can also affect the corrosion rates of buried pipes.

The setup of laboratory pipe-soil tests varies from simple procedures, as documented in ASTM International (2010) and Gupta and Gupta (1979), to relatively complex assemblies (e.g., those shown in Goodman et al., 2013; López et al., 2006, Murray & Moran, 1989; Schwerdtfeger, 1953). Schwerdtfeger (1953) developed a soil corrosion cell that consisted of an electrically connected anode iron and cathode iron made of a material of interest. The anode and cathode iron were separated by soils with different moisture (aeration) to generate a potential difference between them. In
the tests of Gupta and Gupta (1979), mild steel panels (50 mm × 25 mm × 1.6 mm) were buried in soils that were contained in airtight bottles (250 ml). A simple practice to conduct laboratory corrosion tests, under controlled exposure conditions, was documented by ASTM International (2010). In the procedure provided by this standard, small metal specimens are buried in soils fully saturated with 0.1M NaCl solution. Goodman et al. (2013) designed a corrosion cell that was 500 mm in length and 65 mm in diameter, as shown in Figure 2-10. In this corrosion cell, coupons can be placed at the different locations along the sample holder (inner PVC tube) to simulate different buried levels.

![Figure 2-10 Schematic of the test setup in a PVC tube corrosion cell](image)

Source: Goodman et al. (2013).

In comparison, there is little corrosion work on full-scale pipes reported in the literature, except the tests by López et al. (2006) and Murray and Moran (1989). It is to be noted that more reliable data can be produced if the specimens and the soils are prepared close to the actual burial environment. Therefore, it is important to conduct corrosion tests based on real pipe and soil.

In summary, corrosion in buried irons is a complicated process. The formation of corrosion cells in buried pipes varies from case to case, depending on soil properties, pipe installation and geographical characteristics. Of the traditional ways to study corrosion in buried metals, immersion tests are frequently used because the setup process is easy and most of the electrochemical techniques can be conveniently
employed. However, it has been reported that soil-simulated solution tests only weakly replicate the real service soils. Field burial tests take a long time and the results often have great variation due to the heterogeneity of soil and seasonable environment change. In comparison, burial tests in the laboratory are suitable for the quantitative investigation of corrosion in buried pipes. A review of the literature shows that there are a limited number of corrosion tests reported on pipes buried in a controlled soil environment. Therefore, it is necessary to conduct laboratory pipe-soil tests with the aim of further investigating pipe corrosion in soils and extending the current corrosion database concerning buried pipes.

2.4 Factors Affecting Corrosion of Buried Pipes

Soil is a complex and dynamic system. The chemical and physical properties of soils change spatially and seasonally due to precipitation, human activities and plants. To understand corrosion growth in soils under different circumstances, it is necessary to thoroughly examine the effect of each soil property on corrosion behaviour. Section 2.4 specifically reviews the corrosion-influencing factors of buried metal pipes, including soil properties, environmental factors and material metallurgical characteristics.

2.4.1 Water Content

Water content is widely acknowledged as having an important influence on corrosion. Generally, corrosion rates of metal buried in soils with moderate moisture are higher than metal buried in extremely dry or fully saturated soils (Gupta & Gupta, 1979; Kreysa & Schütze, 2008). At low water content, iron is rapidly oxidised into a barrier or protective film that prohibits the diffusion process of water and oxygen (Petersen & Melchers, 2012). However, a higher level of water content can prompt the migration of ferrous ions from the metal surface to soil before being oxidised and accumulate on the surface (Gupta & Gupta, 1979; Petersen & Melchers, 2012; Romanoff, 1957; Rossum, 1969). It can also promote a corrosion reaction by lowering the resistivity of soil. In fully saturated soils, the corrosion process may cease, as the water covers the soil and leads to a deficiency of oxygen (Kreysa & Schütze, 2008). According to Kreysa and Schütze (2008), water content has more influence on the corrosion of iron than many other factors (e.g., salt, organic content
and acidity). The influence of water content (weight %) on corrosion rates of buried iron can be illustrated by the corrosion current curves versus exposure days in Pt/Fe soil cell, as shown in Figure 2-11. Gupta and Gupta (1979) found that there was critical water content (approximately 65% of its water-holding capacity) in soil with maximum corrosion rates. However, in Murray and Moran’s (1989) test, this critical value was not found, probably because a relatively narrow water content range was used. In the field, the water content in soil also changes continually as it is a function of soil type, climate and geometric conditions (Pritchard et al., 2013).

![Figure 2-11 Schematic diagram showing the current versus time curve of the Pt/Fe cell in soils with (a) water content = 10%, (b) water content > 10% and (c) saturated with water (Kreysa & Schütze, 2008)](image)

2.4.2 Soil Resistivity

As corrosion is an electrochemical reaction, soil resistivity plays a major role in determining the corrosion current (Doyle et al., 2003; Flitto & Escalante, 2003; Romanoff, 1957, 1964). Alamilla et al., (2009) found that the ductile and cast iron pipes with the highest corrosion rates were buried in soils with low resistivity (Alamilla et al., 2009). Doyle et al. (2003) suggested that soil resistivity was the only factor that correlated with the maximum pitting depth of buried metal pipes. Soil resistivity is often used to evaluate the corrosivity of soils (e.g., Flitto & Escalante, 2003; Romanoff, 1964). There is a widely accepted relationship between soil resistivity and corrosivity, which was proposed by Roberge (2007) and is displayed in Table 2-2. However, the effect of soil resistivity on corrosion behaviour has been
subjected to debate in the scientific community (Cole & Marney, 2012). For example, Logan et al. (1937) found a very weak correlation between soil resistivity and pit depth. More recently, Petersen and Melchers (2012) claimed that resistivity of soil had an effect on macro corrosion cells, which are built over a long distance in the bulk of the soil; however, it had no effect on micro corrosion cells, formed by the non-homogeneity of soil. In most cases, the analysis of the effect of soil resistivity on corrosion is often complicated, as other secondary factors (such as moisture, soil porosity, salt content and environmental temperature) greatly affect resistivity (Doyle et al., 2003; Moore & Hallmark, 1987; Nürnberger, 2012).

<table>
<thead>
<tr>
<th>Soil Resistivity, Ω cm</th>
<th>Corrosivity Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt; 20,000</td>
<td>Essentially non-corrosive</td>
</tr>
<tr>
<td>10,000–20,000</td>
<td>Mildly corrosive</td>
</tr>
<tr>
<td>5,000–10,000</td>
<td>Moderately corrosive</td>
</tr>
<tr>
<td>3,000–5,000</td>
<td>Corrosive</td>
</tr>
<tr>
<td>1,000–3,000</td>
<td>Highly corrosive</td>
</tr>
<tr>
<td>&lt; 1,000</td>
<td>Extremely corrosive</td>
</tr>
</tbody>
</table>


### 2.4.3 Soil pH

The pH value of soil is known to affect corrosion reaction by acting as a reducing agent in the electrode reaction and influencing the corrosion cell potential (Marcus, 2011). The pH of soil is often measured by a pH meter or electrode to quantify the acidity and hydrogen ion concentration, according to the ASTM International’s (2012b, 2013b) standards. Previous studies on the corrosion of buried pipes showed that the corrosion rate will experience marked increase when pH decreases from four to three, while the corrosion rate does not appear to reply on pH when soil pH is over five (Kreysa & Schütze, 2008; Petersen & Melchers, 2012; Nesic et al., 1996). According to Silverman (2003), in conditions with pH less than five, the corrosion rate is not only related to pH value, but also related with the counter-ions (e.g., SO$_4^{2-}$ and Cl$^-$). An empirical relationship, showing the dependence of corrosion rate ($r$) on the concentration of hydrogen ion ($C_{H^+}$) can be presented as follows (Silverman, 2003):
In Equation 2.8, \( k \) and \( n \) are constants. The value and sign of \( n \) depend on the types of acid. Although the empirical relationship of Equation 2.8 can be observed in a solution, the effect of pH on the corrosion of buried pipes is complex and uncertain in most cases. Also, the pH of soil is affected by many variables (e.g. content of carbon dioxide, organic acid, minerals and contamination by industry wastes; Kreysa & Schütze, 2008). Generally, corrosion behaviour is slower in neutral or alkaline soils (pH from 5.5 to 8.5), except in the presence of microorganisms, such as SRB (Doyle et al., 2003).

2.4.4 Soil Texture

Soil texture is acknowledged as an important factor that influences the diffusion of gases and salts and the movement of corrosion products in soil, which indirectly affect corrosion processes of buried iron (Flitton & Escalante, 2003). In general, soils with finely dispersed structures can retain moisture more readily than other soils (Pritchard et al., 2013). This not only significantly reduces the resistivity of soil, it also promotes the diffusion and migration of corrosion products outward into surrounding soil. This will allow a high rate of corrosion reaction to be maintained (Denison & Romanoff, 1954; Rossum, 1969). Also, a soil with a finely dispersed structure is more likely to be anaerobic, providing SRB with suitable living conditions, which indirectly accelerates corrosion reaction (Doyle et al., 2003). Sandy soils have a large particle size and tend to increase the aeration and movement of water and gases within soil (Flitton & Escalante, 2003). According to Doyle et al. (2003), clay soils often have a high content of dissolved ions in the pore water and can directly facilitate the corrosive reaction. Further, soils with a high content of clay and silt are expected to shrink and crack during drought conditions, allowing oxygen to access the metal substrate and thus, increasing the corrosion rate (Pritchard et al., 2013).

2.4.5 Pipe Material

The element compositions and metallurgical characteristics of pipe material influence its corrosion activation. Most water pipes are made with cast iron, ductile
iron and steel materials, which vary in element compositions and microstructures. It is known (Romanoff, 1957) that the same corrosion process occur in the ferrite phase of the three materials, with the exception that graphitic corrosion can occur in cast iron and ductile iron. This is caused by the fact that the ferrite phase in cast iron and ductile iron can be corroded with respect to the graphitic phase, because of their different corrosion activations (Romanoff, 1957). According to Szeliga and Simpson (2001), cast iron is theoretically prone to being corroded along the margins of graphite flakes, resulting in deeper corrosion penetration than ductile iron, which has dispersed graphite nodules. However, the field burial tests by the US NBS showed that there was no significant difference in the pitting between cast iron and ductile iron in the same environments (Romanoff, 1964). Compared with the variations in materials, it was found that the properties of soils have more impact on the corrosion behaviour of buried pipes (Kreysa & Schütze, 2008, Romanoff, 1964, 1957).

2.4.6 Environmental Factors

Environmental factors (e.g., temperature, sulphate-reducing organisms and stray current) are known to directly or indirectly promote the corrosion of buried pipes, in addition to soil and pipe material properties (Davis, 2000; McDougall, 1966; Nie et al., 2009; Usher et al., 2014). For example, according to Davis (2000), the corrosion rate of iron can increase approximately two times if the temperature increases by 10 °C. It is also known that temperature affects soil resistivity, solubility of oxygen in soil pore water, oxidation reaction of ferrous iron and the property of protective film (Flitton & Escalante, 2003; Pritchard et al., 2013; Nesic et al., 1996; Nie et al., 2009). According to Romanoff (1957), with temperatures ranging from 0 °C to 25 °C, the dependence of resistivity of temperature is given by Equation 2.9:

\[ R_t = \frac{40R_{15.5}}{(24.5 + t)} \]  
(Equation 2.9)

In this equation, \( R_t \) (ohm-cm) is the resistivity of soil at temperature \( t \), \( R_{15.5} \) is the resistivity of soil at 15.5 °C. However, it is worth noting that an increase in temperature can prompt corrosion reaction, but the increase of temperature in the soil can also result in evaporation and loss of moisture, slowing the corrosion rate due to moisture loss. Further, the variation of soil atmospheric pressure was found to affect corrosion by differing the concentrations of soils gases (especially \( \text{CO}_2 \) and \( \text{O}_2 \)).
throughout the soil substrate (Pritchard et al., 2013). Moreover, the presence of sulphate-reducing organisms can accelerate corrosion in buried pipes (Davis, 2000).

In summary, there are many factors that affect the corrosion in buried pipes (e.g., water content, soil resistivity, soil pH, salts concentration and temperature). Generally, soils with poor aeration, high electrical conductivity and moisture content and high levels of soluble salts are recognised as corrosive soils. The environmental factors (e.g., temperature, atmospheric pressure and sulphate-reducing organisms) also play a significant role in the corrosion of buried pipes, but their effects are often difficult to determine. The analysis of these corrosion-influencing factors is often complicated by the interaction between the solid, liquid and gas phases. Further, most of the corrosion-influencing factors affect each other and some of them (e.g., temperature and moisture) can impose opposite effects on the corrosivity of soil. As a result, it can be difficult to determine a single corrosion-influencing factor. Considering these limitations, it is necessary to investigate the corrosion-influencing factors to determine their interactions and obtain a comprehensive understanding of the relationship between corrosion-influencing factors and corrosion behaviour.

### 2.5 Effect of Corrosion on Metal Pipes

To evaluate the integrity of corroded pipelines and predict its failure possibility, knowledge concerning the corrosion effect on pipe material and mechanical properties is necessary. The corrosion effects on buried metal pipes that are reviewed in Section 2.5 include stress-induced strength decreases at a macro level and corrosion-induced material changes at a micro level. Both experimental and numerical tests on corroded pipelines are specifically reviewed.

#### 2.5.1 Effect of Corrosion on Pipe Strength

Corrosion is expected to cause the strength of pipe structures to decrease because of cross-section loss. It can also cause strength degradation through a far more complicated phenomenon: stress redistribution and concentration. Research concerning the corrosion effect on stress distribution of structures can be dated to 1913, when Inglis (1913) found that a circular defect in a flat plate can cause an intensified stress at the edge three times of the far-field stress. For a pressurised pipe, the presence of a corrosion defect can force the hoop stress redistribution and cause
stress concentration around the corrosion (Ahammed, 1998). If the intensified stress at the corroded pipe is greater than the stress limit of the material, failure eventually occurs (Ahammed, 1998). The influence of corrosion on pipes can be determined by considering the corrosion dimensions, pipe material and loading conditions. According to Benjamin et al., (2007), the interactive effect of close corrosion pits in pipes can further lower its residual strength. Other factors, such as fatigue, can also affect the residual strength of pipes (Belmonte et al., 2009). Overall, corrosion can reduce the strength of pipes in two ways: the reduction of residual strength, due to local wall thinning; and redistribution of stress and stress concentration around the corrosion site.

2.5.2 Effect of Corrosion on Material Change

The materials can be identified according to the element compositions and the characteristics of the microstructure. For example, cast iron and ductile iron are generally characterised by an alloy of carbon content over 2% and silicon content of approximately 1–3%, while steel has less than 1.2% by weight of carbon (Keil & Devletian, 2011; Thomson & Wang, 2009). Steel is typically dominated by the pearlite (Thomson & Wang, 2009). Cast iron is normally characterised by the presence of graphite flakes in the matrix of pearlite or ferrite (Rajani, 2000), while in ductile iron the graphite are separately dispersed nodules (Szeliga & Simpson, 2001).

Corrosion can change the elemental composition of materials and/or its microstructure, resulting in material deterioration and the degradation of mechanical properties. Generally, the materials of ferrous pipes differ from each other in the content of iron, carbon and other elements (such as silicon, manganese, phosphorus, sulphur and chromium). These elements normally have different corrosion activations and the active components are subject to preferential corrosion or dissolution, also known as selective corrosion, leaving the noble components (Ateya et al., 2014). The elements in iron alloys have various effects on mechanical properties of metal, as illustrated in Table 2-3. The selective corrosion of certain elements can cause the change of composition portions and eventually alter the mechanical properties of bulk materials. Further, selective corrosion can induce a porous structure at the top surface and invite ingress of oxidants (e.g., oxygen may accelerate corrosion and material deterioration; Ateya et al., 2014; Hou et al., 2016).
One example is the preferential leaching of iron from the iron-graphite matrix in cast iron, also known as graphitic corrosion (Petersen & Melchers, 2012). Another example is the preferential corrosion grain boundaries in steel, as shown in Figure 2-12, known as intercrystalline corrosion (Kaesche, 2011). This type of corrosion can lead to the development of grain boundary trenches and disintegration, causing great strength loss of material (Kaesche, 2011).

### Table 2-3 Summary of the effect of elements on mechanical properties of iron alloys

<table>
<thead>
<tr>
<th>Element</th>
<th>Influence</th>
</tr>
</thead>
<tbody>
<tr>
<td>Iron (Fe)</td>
<td>Increase the ductility of the material</td>
</tr>
<tr>
<td>Carbon (C)</td>
<td>Increase the strength and hardness of metal, but decrease ductility</td>
</tr>
<tr>
<td>Silicon (Si)</td>
<td>Increase the strength and hardness of material, but less than that of manganese</td>
</tr>
<tr>
<td>Phosphorus (P)</td>
<td>Increase strength &lt; 0.1%, while increase brittleness if &gt; 0.1%</td>
</tr>
<tr>
<td>Sulphur (S)</td>
<td>Increase the brittleness of material</td>
</tr>
<tr>
<td>Manganese (Mn)</td>
<td>Increase the strength and hardness of metal, but less than that of carbon</td>
</tr>
<tr>
<td>Chromium (Cr)</td>
<td>Increase the hardness of material greatly; increase the yield strength</td>
</tr>
<tr>
<td>Nickel (Ni)</td>
<td>increase the toughness of material</td>
</tr>
</tbody>
</table>

Source: Park et al., (2009).

### 2.5.3 Experiments on Pipes

To understand the effect of corrosion on the structural integrity of pipes, mechanical tests based on strength theory are commonly employed. The mechanical tests reviewed in Section 2.5.3 include tension tests, crush ring and burst tests and FE tests.
Figure 2-12 Schematic of intercrystalline corrosion in stainless steel


2.5.3.1 Tension Test

Tension tests are a fundamental method for determining the mechanical properties of materials. The application of tension tests on determining the strength of pipes can be dated to the 1930s when Talbot (1926) tested the tensile strengths of specimens machined from cast iron pipes free of corrosion. Later, tension tests were often used to determine the effect of corrosion on the residual strength of materials. For example, Yamamoto et al., (1983) investigated the residual strength of several cast iron pipes with various degrees of corrosion, which were characterised by the percentage of the graphitisation area over the cross-section of pipe wall. The test results showed that generally, the peak loads at failure decreases when the degree of corrosion increases. Seica and Packer (2004) conducted tension tests on specimens cut from over 100 ex-service cast iron pipes. The results also showed that corrosion generally resulted in the loss of strength. Kim et al., (2007) and Atkinson et al., (2002) conducted similar tension tests on several exhumed cast iron pipes. The test results of Kim et al. (2007) indicated that the ratio of the averaged pit depth to pipe wall thickness can be reasonably correlated to its residual tensile strength. Similarly, Atkinson et al. (2002) found that the ratio of the residual strength to the corrosion pit depth correlated with the material section loss. Garbatov et al., (2014) tested the yield strength and tensile strength of steel that had experienced accelerated corrosion in sea water. A nonlinear relationship between mass loss (defined as the degree of
degradation) and tensile strength was obtained, as shown in Figure 2-13. In the tests of Hou et al. (2016), a similar relationship between the reduction of strength and mass loss was found in both steel and cast iron.

![Graph showing reduction of tensile strength with respect to corrosion-induced degradation](image)

**Figure 2-13 Reduction of tensile strength with respect to corrosion-induced degradation**

Source: Garbatov et al. (2014).

### 2.5.3.2 Crush Ring Test

Crush ring tests are normally used to characterise the loading capacity and determine the modulus of rupture of pipes (Rajani, 2000; Seica & Packer, 2004). A typical setup for a crush ring test can be observed in Figure 2-14a. Although the modulus of rupture can also be determined by a four-point bending test, the preparation of ring tests is relatively cost-effective and easy. The crush ring test is a preferable testing method especially for buried pipes, because the loading condition can simulate the vertical gravitational soil effect on the bending strength of pipe. Like tension tests, crush ring tests were mainly used to characterise the modulus of rupture of pipes for design purpose (e.g., Rajani, 2000; Talbot, 1926). The tests based on corroded pipes are highly limited, except for few. For example, Seica and Packer (2004) performed crush ring tests on corroded cast iron pipes that were excavated from several different locations of Toronto, Ontario to characterise the effect of corrosion on the modulus of rupture of pipes. The test results showed that the difference between the
gross thickness-based modulus of rupture and the net wall thickness modulus of rupture varies from 3% to 43% (Seica & Packer, 2004). However, a relationship between the loss of modulus of rupture and corrosion or time was not established.

---

**Figure 2-14** (a) Typical testing setup for crush ring test (Seica & Packer, 2004) and (b) test apparatus of pipe bursting test (Netto et al., 2005)
2.5.3.3 Burst Test

A burst test is a testing method that is frequently used to evaluate the failure loading in pressurised pipes. A considerable number of studies have been conducted by previous researchers to investigate the effect of corrosion on the residual strength of corroded pipes. For example, Benjamin et al., (2000) conducted burst tests on API X60 steel pipes with an artificial corrosion defect on a long rectangular shape and smooth surface. Choi et al., (2003) and Freire et al., (2006) presented the tests results obtained from API X65 steel pipes subjected to similar blunt corrosion defects. However, it was found that the failure manner of pipes with (artificially made) flat-bottomed defects differed from those in pipes with natural corrosion (Cronin, 2000). In a series of burst tests on pipes by Souza et al., (2007), Freire et al., (2007), Benjamin et al. (2007) and Benjamin et al., (2010), corrosion defects with more complex shapes were machined to approximate real corrosion. The result of these tests showed that the interacting corrosion pits lowered the failure pressure of pipes and pipes could fail at loads lower than that predicted, based on a single pit. A typical test setup for a burst test can be observed in Figure 2-14b. Compared with other tests (e.g., tension and crush ring tests), a great deal of efforts is required during the processes of specimen preparation and testing setup in burst test.

2.5.3.4 Finite Element–Based Tests

The FE method has been frequently used over the last two decades to analyse the stress distribution of corroded pipelines and estimate failure load (e.g., Ahmmad & Sumi, 2010; Choi et al., 2003; Diniz et al., 2006; Ji et al., 2017; Nakai et al., 2006). For example, Ji et al., (2015) and Zhang et al., (2017) investigated the stress concentration factors of pipes that were subjected to corrosion pits, idealised as a semi-ellipsoidal cavity. Ji et al. (2017) further proposed a predictive equation of stress concentration factors for corroded pipes buried in soil, which was simplified as a linear elastic material based on the work of Robert et al., (2016). In these three-dimensional FE analyses, different geometries were assumed for corrosion-induced pits, such as rectangular-shaped box (Vodka, 2015), semi-ellipsoidal cavity (Ji et al., 2015; Kolios et al., 2014), circular cone (Ahmmad & Sumi, 2010; Nakai et al., 2006) and truncated cone (Ji et al., 2017). These corrosion geometrical models may
approximate blunt corrosion well; however, they can result in serious error in the approximation of sharp corrosion pits that can cause high-stress localisation.

In summary, the degradation of mechanical properties in corroded pipes involves two aspects: corrosion-induced stress redistribution and concentration resulted from corrosion pits and section loss, and changes in the chemical compositions and microstructure of material. The mechanical tests that are frequently used to determine the mechanical properties of corroded pipes have been reviewed. These tests used traditional strength theory, rather than fracture mechanics, to determine the effect of corrosion on pipes. Although many burst tests and FE-based simulations have been conducted to investigate the effect of corrosion, the corrosion pits were often assumed to be blunt; sharp corrosion pits were not considered.

2.6 Basics of Fracture Mechanics

To understand fracture-related pipe failure, knowledge of fracture mechanics and its application are needed. In 1920, Griffith was the first person who realised that the internal flaws presented in glass rods had an important effect on fracture failure. Based on experiments, Griffith (1920) defined a relationship between fracture stress and flaw size for brittle materials. Later, Irwin (1948) extended Griffith’s theory after exploring the significant role of plasticity in the fracture of ductile material. The dissipated energy resulting from the local plastic flow was considered. In Section 2.6, the fundamentals of linear elastic fracture mechanics are reviewed. Additionally, the methods for determining SIF and fracture toughness are briefly presented.

2.6.1 Stress Intensity Factor

In 1957, the concept of SIF was proposed by Irwin (1957). This theory has been widely used to quantify the stress field near a crack tip until today. If a polar coordinate is defined with the origin located at the tip of a crack in a linear elastic body, as shown in Figure 2-15, the stress field can be expressed as (Hertzberg, 1996):

\[
\sigma_{ij} = \left(\frac{k}{\sqrt{r}}\right)f_i(\theta) + \sum_{m=0}^{\infty} A_m r^\frac{m}{2} g^{(m)}_i(\theta)
\]  

(Equation 2.10)
In Equation 2.10, $\sigma_{ij}$ is the stress tensor, $k$ is a constant, $r$ and $\theta$ are coordinates of the polar coordinate, $f_{ij}$ and $g^{(m)}_{ij}$ are dimensionless functions of $\theta$ and $A_m$ is the $m^{th}$ term of amplitude. Equation 2.10 defines an asymptotic stress field in which the stress varies with $1/\sqrt{r}$ when $r$ approaches zero, which is also known as stress singularity.

![Figure 2-15 Stresses near a crack tip](image)


To quantify the stress singularity of a cracked body, Irwin (1957) defined the concept of SIF, as shown in Equations 2.11a–c:

$$\lim_{r\to0} \sigma_{ij}^{(I)} = \frac{K_I}{\sqrt{2\pi r}} f_{ij}^{(I)}(\theta) \quad \text{(Equation 2.11a)}$$

$$\lim_{r\to0} \sigma_{ij}^{(II)} = \frac{K_{II}}{\sqrt{2\pi r}} f_{ij}^{(II)}(\theta) \quad \text{(Equation 2.11b)}$$

$$\lim_{r\to0} \sigma_{ij}^{(III)} = \frac{K_{III}}{\sqrt{2\pi r}} f_{ij}^{(III)}(\theta) \quad \text{(Equation 2.11c)}.$$ 

In these equations, I, II, III represent the three different modes of fracture: opening mode (Mode I), in-plane shear mode (Mode II) and out-of-plane shear or tear mode (Mode III), as shown in Figure 2-16. The Mode I fracture mode is commonly recognised as the dominant fracture form in pipes under normal operating conditions (Li & Yang, 2012).
Figure 2-16 Three basic modes of crack displacements


2.6.2 Griffith’s Energy Theory

In terms of defining fracture failure (i.e., formation or growth of a crack), a criterion based on energy balance theory was developed by Griffith (1920). It states that for an increase of crack length, sufficient energy must be stored to overcome the surface energy of the material, which can be simply expressed as follows (Anderson, 2017):\[ \frac{dE}{da} = \frac{d\Pi}{da} + \frac{dW_s}{da} = 0 \] (Equation 2.12)

In Equation 2.12, \( da \) is an incremental increase of crack length, \( E \) is total energy, \( \Pi \) is potential energy and \( W_s \) is work required to create new surfaces. Based on this energy-based criterion, the fracture stress of a cracked body can be calculated. Examples of this have been provided by Anderson (2017).

2.6.3 J-Contour Integral

In 1956, Irwin defined the concept of the energy release rate, \( G \), to measure the energy required for the extension of crack length, as follows (Irwin, 1956):

\[ G = -\frac{d\Pi}{da} \] (Equation 2.13)

Rice (1968) proposed a path-independent contour integral, called \( J \), to calculate the energy release rate of a cracked nonlinear elastic material, as follows:
\[ J = - \frac{d\Gamma}{da} = \int_{\Gamma} \left( W_s dy - \bar{T} \cdot \frac{\partial \bar{u}}{\partial x} ds \right) \]  

(Equation 2.14)

In Equation 2.14, \( \Gamma \) is the contour that encloses the crack tip, \( W_s \) is the strain energy density, \( ds \) is the increment of the contour path, \( \bar{T} \) is the outward traction vector on \( ds \) and \( \bar{u} \) is the displacement vector at \( ds \). Physically, \( J \) is equivalent to \( G \) in characterising the energy required for an extension of a crack in material. For linear elastic material, the SIF can be related to the energy release rate, \( G \), and \( J \)-contour integral as follows (Hertzberg, 1996):

\[ K = \sqrt{GE} = \sqrt{JE} \]  

(Equation 2.15)

In Equation 2.15, \( \bar{E} = E \) represents plane stress problem and \( \bar{E} = E/(1 - \nu^2) \) represents plane strain problem and \( \nu \) is Poisson’s ratio.

### 2.6.4 Methods of Determining Stress Intensity Factor

To determine the SIF (\( K \)) and its critical value (\( K_C \)), numerical methods and experimental tests are commonly used. This section reviewed these two aspects for Mode I fracture mode in respect to linear elastic material.

#### 2.6.4.1 J Integral–Based Numerical Approach

Based on fracture mechanics, the SIF (\( K \)) of a cracked structure can be expressed in a function of the applied stress, crack size and dimensions of the system that contains the crack, as follows (Hertzberg, 1996):

\[ K_I = \sigma_0 \sqrt{\pi a f} \]  

(Equation 2.16)

In this equation, \( K \) is SIF, \( \sigma_0 \) is the applied stress, \( a \) is the crack size and \( f \) is the influence of the coefficient function that depends on the geometries of structure and crack. For a structure with complex geometry, it is widely accepted that it is extremely difficult to derive the SIF in an analytical way. The \( J \) integral–based numerical approach has been proved to have a reasonable accuracy (e.g., Atluri & Kathiresan, 1980; Barsoum, 1974; Li & Yang, 2012; Raju & Newman, 1982). In this method, the key is to obtain the influence of the coefficients of function \( f \) and the SIF
$K$ to calculate for linear elastic material (see Equation 2.15). For a two-dimensional cracked body, as illustrated in Figure 2-17, with the stresses at the Gauss points known, all components in Equation 2.14 can be determined as follows (Mohammadi, 2008):

$$ W = \frac{1}{2} \left[ \sigma_{xx} \frac{\partial u_x}{\partial x} + \sigma_{yy} \frac{\partial u_y}{\partial y} + \sigma_{xy} \left( \frac{\partial u_x}{\partial y} + \frac{\partial u_y}{\partial x} \right) \right] $$

(Equation 2.17a)

$$ \bar{T} \frac{\partial u}{\partial x} = \left[ (\sigma_{xx} n_1 + \sigma_{xy} n_2) \frac{\partial u_x}{\partial x} + (\sigma_{yy} n_1 + \sigma_{xy} n_2) \frac{\partial u_y}{\partial x} \right] $$

(Equation 2.17b)

$$ ds = \sqrt{\left( \frac{\partial x}{\partial \eta} \right)^2 + \left( \frac{\partial y}{\partial \eta} \right)^2} \, d\eta $$

(Equation 2.17c)

Therefore, substituting Equation 2.17a–c into Equation 2.14 results in Equation 2.18 (Mohammadi, 2008):

$$ J = \left\{ \begin{array}{l}
\frac{1}{2} \left[ \sigma_{xx} \frac{\partial u_x}{\partial x} + \sigma_{yy} \frac{\partial u_y}{\partial y} + \sigma_{xy} \left( \frac{\partial u_x}{\partial y} + \frac{\partial u_y}{\partial x} \right) \right] \frac{\partial y}{\partial \eta} \\
- \left[ (\sigma_{xx} n_1 + \sigma_{xy} n_2) \frac{\partial u_x}{\partial x} + (\sigma_{yy} n_1 + \sigma_{xy} n_2) \frac{\partial u_y}{\partial x} \right] \sqrt{\left( \frac{\partial x}{\partial \eta} \right)^2 + \left( \frac{\partial y}{\partial \eta} \right)^2}
\end{array} \right\} d\eta $$

(Equation 2.18)
2.6.4.2 Experimental Test

To determine the fracture toughness of material, $K_C$ (i.e., the critical SIF), of metallic materials, the testing methods can be found in standards (e.g., ASTM International, 2012a). Typically, the prenotched specimens are first loaded in fatigue tests to create a sharp front. Then, the pre-cracked specimens are loaded until failure occurs (i.e., unstable crack extension) due to a tension or three-point bending test, from which the crack mouth opening displacement (CMOD) is measured against the force used. From the CMOD force curve, the provisional force, $P_Q$, is determined according to the standards. Fracture toughness, $K_C$, is correspondingly calculated by the equations that have been developed for the specimen with specific configurations of stress analysis. For example, a typical setup of the three-point bending test can be observed in Figure 2-18. The Mode I fracture toughness of material can be determined in this test setup by the following equations (ASTM International, 2012a):

$$K_C = \frac{P_Q S}{BW^{3/2}} f\left(\frac{a}{W}\right)$$  \hspace{1cm} (Equation 2.19)

$$f\left(\frac{a}{W}\right) = \frac{3\left(\frac{a}{W}\right)^{1/2}}{2(1+2\frac{a}{W})(1-\frac{a}{W})^{3/2}} \left[ 1.99 - \left(\frac{a}{W}\right)(1-\frac{a}{W})(2.15 - 3.93\left(\frac{a}{W}\right) + 2.7\left(\frac{a}{W}\right)^2) \right]$$
In Equations 2.19 and 2.20, \( S \) is support span, \( B \) is specimen thickness, \( W \) is specimen width and \( a \) is crack size.

![Figure 2-18 Typical setup of a three-point bending test](image)


In summary, Section 2.6 has outlined the basics of fracture mechanics. The SIF, as a significant parameter that characterises the stress singularity of cracked material, is a function of the applied stress and the configurations of specimens. For linear elastic material, the energy release rate \( (G) \) equals to \( J \), which has been successfully applied to characterise the state of nonlinear material. The numerical method based on the \( J \)-contour integral is known to be accurate in calculating the SIFs of structures with complicated configurations. The methods for determining the fracture toughness of material has been illustrated.

### 2.7 Assessment of Buried Pipelines

To accurately predict the remaining safe life of corroded pipelines, it is necessary to integrate knowledge about material property, deterioration models and reliability theory into a methodology to analyse the behaviour and failure mechanisms of pipes subjected to corrosion and loading. This section will critically review these important aspects.
2.7.1 Pipe Failure Models

Over the last few decades, many works have been conducted to establish an expression of failure stress as a function of the loading, material properties, dimensions of defects and pipes (e.g., Cronin, 2000; Kiefner & Vieth, 1989; Mok et al., 1991). The previous works can be broadly categorised into small-scale material tests, with no corrosion considered, and mechanical properties tests on pipes with corrosion defects. Both of these tests are important in constructing a holistic understanding of failure mechanisms in corroded pipes.

2.7.1.1 Material Failure Criterion

A series of failure criterion for brittle and ductile materials have been developed. The first investigation concerning failure in metal occurred during the years 1820–1872, when William Rankine developed the first failure criterion for brittle material, called Rankine criterion or maximum principal stress criterion. In 1864, Tresca developed the yield criterion for metal. This yield criterion is known as Tresca criterion or maximum shear-stress criterion. Tresca criterion states that failure will commence if the maximum shear-stress exceeds a specific threshold. Von Mises (1913) proposed another failure criterion for ductile material, known as distortional energy density criterion. Considering the dissimilar mechanical properties of cast iron in tension and compression, a failure criterion for cast iron was first developed by Fisher (1952), who modified the distortion energy criterion and approximated the graphite flake embedded as an ellipsoid-shaped internal cavity. Coffin (1950) later conducted tests on cast iron tubes under various tensile and compressive stress ratios and developed another failure criterion by introducing a residual stress at the edge of flakes. Mair (1968) conducted a series of similar tests and the results demonstrated agreement with the predictions of Coffin’s criterion. These criteria constitute the underpinning conceptual theory of the mechanical properties in metal and are summarised in Table 2-4.
Table 2-4 Summary of common material failure criteria for metal

<table>
<thead>
<tr>
<th>Name</th>
<th>Expression</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rankine criterion</td>
<td>$\sigma_1 - \sigma_y = 0$</td>
</tr>
<tr>
<td>Tresca criterion</td>
<td>$\sigma_1 - \sigma_3 = \sigma_T$</td>
</tr>
<tr>
<td>Distortional energy density</td>
<td>$\sqrt{\frac{1}{2}[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2]} = \sigma_y$</td>
</tr>
<tr>
<td>Fisher’s criterion</td>
<td>Biaxial tension: $K\sigma_1 = \sigma_y$</td>
</tr>
<tr>
<td></td>
<td>Tension-compression: $(K\sigma_1)^2 - K\sigma_1\sigma_2 + \sigma_2^2 = \sigma_y^2$</td>
</tr>
<tr>
<td></td>
<td>Biaxial compression: $(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2 = \sigma_y^2$</td>
</tr>
<tr>
<td>Coffin’s criterion</td>
<td>$K\sigma_1 + \sigma_0 = \sigma_y$</td>
</tr>
<tr>
<td></td>
<td>$(K\sigma_1 + \sigma_0)^2 - K\sigma_1\sigma_2 + \sigma_2^2 = \sigma_y^2$</td>
</tr>
<tr>
<td></td>
<td>$\tau_n + \mu\sigma_n = M$</td>
</tr>
</tbody>
</table>

Note: $\sigma_1$, $\sigma_2$, $\sigma_3$ are major, intermediate and minor stresses, respectively. $\sigma_T$ is tensile stress at failure, $\sigma_y$ is yield strength, $K$ is approximately 3, $\tau_n$ is the critical (failure) shearing stress, $\mu$ and $M$ are constants depending on materials and $\sigma_0$ is a residual stress.

2.7.1.2 Failure Load of Cast Iron Pipes

The model that has been most widely used in the assessment of cast iron pipes is Schlick’s failure criterion, which defines the occurrence of failure if the following equation is satisfied (Schlick, 1940):

$$\left(\frac{W}{W_L}\right)^2 + \frac{P}{P_L} > 1$$  \hspace{1cm} (Equation 2.21)

In Equation 2.21, $W$ and $P$ are vertical load and internal pressure, respectively. $W_L$ and $P_L$ are critical vertical load and internal pressure, respectively. The exponent for normalised load, $W/W_L$ in Equation 2.21, is determined empirically. A schematic of Schlick’s failure criterion is presented in Figure 2-19. $W_L$ and $P_L$ can be estimated through the modulus of rupture ($\sigma_r$) and ‘bursting tensile strength’ ($\sigma_b$) determined from crush ring and burst test, as follows (Rajani & Makar, 2000):
Figure 2-19 Schematic of Schlick’s failure criterion (modified from Robert et al., 2016)

This failure criterion was adopted by American Water Works Association’s (1977) pipe design code and has been widely used in failure assessments of rigid pipes, such as cast iron and concrete pipes (e.g., Camarinopoulos et al., 1999; Fahimi et al., 2016; Rajani & Abdel-Akher, 2012; Robert et al., 2016; Watkins & Anderson, 1999). In Schlick’s failure criterion, only in-plane direction loads are considered. However, the failures of small diameter pipes (less than 150 mm in diameter) occur in the form of circumferential breaks, which are associated with stresses in the longitudinal direction due to bending loads or temperature change (Makar et al., 2001). Therefore, this criterion theoretically cannot be used in the assessment of small pipes. Conlin and Baker (1991) proposed an equation for predicting the failure moment of corroded cast iron pipes as follows:

\[ M_f = 0.156\sigma_i A_i \left( D_o^2 + D_i^2 \right)/D_o \]  

(Equation 2.24)

In Equation 2.24, \( \sigma_i \) is the uniaxial tensile strength, \( A_i \) is the corroded cross-section area, and \( D_o \) and \( D_i \) are the original external diameter and internal diameter, respectively. The development of Equation 2.24 was based on the assumption that a
pipe contains blunt corrosion (Conlin & Baker, 1991). For pipes subjected to sharp notch-like corrosion, the failure moment (based on fracture mechanics) has been developed by Conlin and Baker (1991):

\[ M_f = 0.125 \frac{K_q}{Y \sqrt{\pi a}} A \left( D_0^2 + D_i^2 \right) / D_0 \]  
(Equation 2.25)

In Equation 2.25, \( K_q \) is the provisional fracture toughness, \( Y \) is the geometric function and \( a \) is corrosion depth. The work of Conlin and Baker (1991) was the first of its kind; they applied fracture mechanics to the analysis of pipes with crack-like defects. More recently, Rajani (2000) developed an expression of residual tensile strength, incorporating corrosion dimensions and the concept of fracture mechanics:

\[ \sigma_n = \frac{\alpha K_q}{Y \left[ a / (d \sqrt{a_n}) \right]} \]  
(Equation 2.26)

In Equation 2.26, \( \sigma_n \) is the nominal tensile strength, \( d \) is pipe wall thickness, \( a_n \) is the size of corrosion pit and \( a \) and \( s \) are constants obtained from data fit.

### 2.7.1.3 Failure Load of Steel Pipes

The traditional methods that predict the failure load of corroded pipelines are documented in various codes and standards (e.g., American Society of Mechanical Engineers & American National Standards Institute, 1985; Anderson & Osage, 2000; British Standards Institution, 2005; Canadian Standard Association, 2007; Det Norske Veritas, 1999). Among these standards, B31G (American Society of Mechanical Engineers & American National Standards Institute, 1985) is the most widely used method, in which the projected areas of corrosion pits are approximated by a parabola. The B31G solution was modified by Kiefner and Vieth (1989); improvements included material flow stress, a new defined Folias factor and an iterative method of calculating the corrosion area. DNV-RP-F101 (Det Norske Veritas, 1999) was developed based on pipes with rectangular-shaped corrosion defects. In CSA Z662 (Canadian Standard Association, 2007), depending on the length of corrosion, the failure pressure is distinguished as burst pressure and rupture. These models are summarised in Table 2-5.
However, it has been found that the models documented in these standards often produce excessively conservative results (Benjamin et al., 2000; Benjamin et al., 2007; Benjamin et al., 2010; Choi et al., 2003; Freire et al., 2007; Souza et al., 2007). Therefore, some other models have been developed. For example, Cronin (2000) proposed an equation to predict the failure pressure of a pipe with a longitudinally oriented groove. Choi et al. (2003) developed a solution of the maximum allowable pressure for API X65 steel pipes, with both rectangular and elliptical shaped pits. A good agreement was achieved when the results were compared with experimentally determined failure pressure (Choi et al., 2003). Cronin and Pick (2002) used weighted depth difference to quantify the size of corrosion defects and their interactions. It was found that the models of Cronin and Pick (2002) produced less conservative results, compared with the methods codes and standards. These models are also summarised in Table 2-5.

To evaluate the failure load of steel pipes subjected to crack flaws, the failure assessment diagram (FAD) method is used, as documented in standards codes, including API Recommended Practice 579 (Anderson & Osage, 2000), BS 7910 (British Standards Institution, 2005) and EDF Energy (2001). The FAD method works by plotting the load ratio \( L_r \) against the toughness ratio \( K_r \), as shown in the curve of Figure 2-20. If the stresses of the structure fall within the curve, the flaw is acceptable and vice versa. This failure criterion is written as follows (Anderson & Osage, 2000):

\[
K_r = \left( 1 - 0.14L_r^2 \right) \left( 0.3 + 0.7e^{-0.65L_r} \right) \tag{Equation 2.27}
\]

\[
L_r = \frac{\sigma_{\text{ref}}}{\sigma_y} \tag{Equation 2.28}
\]

\[
K_r = \frac{K_P^p + \phi K_{SR}^P}{K_{\text{mat}}} \tag{Equation 2.29}
\]

In Equations 2.27–2.29, \( \sigma_{\text{ref}} \) is reference stress, \( \sigma_y \) is yield stress, and \( K_P^p \) and \( K_{SR}^P \) are SIFs due to main load and secondary/residual stress, respectively. \( \phi \) is an adjustment factor.
Table 2-5 Summary of the failure pressure of steel pipes subjected to corrosion defects

<table>
<thead>
<tr>
<th>Source</th>
<th>Expression</th>
</tr>
</thead>
<tbody>
<tr>
<td>B31G (American Society of Mechanical Engineers &amp; American National Standards Institute, 1985)</td>
<td>( P_f = 1.1 \sigma_{SMYS} \frac{d}{R} \left[ 1 - \frac{2}{3} \left( \frac{a_{\max}}{d} \right) \right] ) for ( \frac{l}{\sqrt{2Rd}} \leq 4.479 ), ( M = \sqrt{1 + \frac{0.8l^2}{2Rd}} )</td>
</tr>
</tbody>
</table>
| B31G (American Society of Mechanical Engineers & American National Standards Institute, 1985) | \( P_f = 1.1 \sigma_{SMYS} \frac{d}{R} \left[ 1 - \frac{a_{\max}}{d} \right] \) for \( \frac{l}{\sqrt{2Rd}} > 4.479 \)
| DNV-RP-F101 (Det Norski Veritas, 1999) | \( P_f = \sigma_{UTS} \frac{2d}{R} \left( 1 - \frac{a}{d} \right) \right) \) for \( \frac{l}{\sqrt{2Rd}} \leq 4.479 \), \( M = \sqrt{1 + \frac{0.31l^2}{2Rd}} \) |
| Modified B31G (Kiefner & Vieth, 1989) | \( P_f = (\sigma_{SMYS} + 10000) \frac{d}{R} \left[ 1 - \left( \frac{A}{A_0} \right) \left( 1 - \frac{A}{A_0} M^{-1} \right) \right] \)
| Modified B31G (Kiefner & Vieth, 1989) | \( M_R = \sqrt{1 + 0.6275 \frac{l^2}{2Rd} - 0.003375 \frac{l^4}{4R^2d^2}} \) for \( \frac{l^2}{2Rd} \leq 50 \)
| Modified B31G (Kiefner & Vieth, 1989) | \( M_R = 0.032 \frac{l^2}{2Rd} + 3.3 \) for \( \frac{l^2}{2Rd} > 50 \)
| CSA Z662 (Canadian Standard Association, 2007) | \( P_{rupture} = \frac{0.9d\sigma_{UTS}}{M_R R} \)
| CSA Z662 (Canadian Standard Association, 2007) | \( P_{burst} = e_1 \left( 1 - \frac{a_{\text{avg}}}{d} \right) + (1 - e_1) \frac{0.9d\sigma_{UTS}}{R} - e_2\sigma_{UTS} \) \( M_R \) in CSA Z662 same to \( M_R \) in Modified B31G |
For $\frac{l}{\sqrt{R_d}} < 6$

$$P_f = \frac{0.9d\sigma_{UTS}}{R_i} \left[ C_2 \left( \frac{l}{\sqrt{R_d}} \right)^2 + C_1 \frac{l}{\sqrt{R_d}} + C_0 \right]$$

$$C_2 = 0.1163 \left( \frac{a}{d} \right)^2 - 0.1053 \frac{a}{d} + 0.0292$$

$$C_1 = -0.6913 \left( \frac{a}{d} \right)^2 + 0.4548 \frac{a}{d} - 0.1447$$

$$C_0 = 0.06 \left( \frac{a}{d} \right)^2 - 0.1035 \frac{a}{d} + 1$$

For $\frac{l}{\sqrt{R_d}} \geq 6$

$$P_f = \frac{d\sigma_{UTS}}{R_i} \left[ C_1 \frac{l}{\sqrt{R_d}} + C_0 \right]$$

$$C_1 = 0.0071 \frac{a}{d} - 0.0126$$

$$C_0 = -0.9847 \frac{a}{d} + 1.1101$$

Figure 2-20 Failure assessment diagram

Source: Anderson & Osage (2000).
2.7.2 Reliability Methods

To predict the failure probabilities of pipes subjected to corrosion, reliability methods—based on probability theory—are used for civil engineering infrastructures. A review of the literature shows that the methods vary, from traditional time-independent reliability methods to more advanced time-dependent reliability methods (Camarinopoulos et al., 1999; Li & Mahmoodian, 2013; Melchers, 1999; Sadiq et al., 2004; Yamini, 2009). Three of the frequently used methods, (i.e., integration method, Monte Carlo simulation and first-order second-moment method (FORM) are discussed in Section 2.7. A time-dependent approach, based on upcrossing theory, is critically reviewed.

2.7.2.1 Integration Method

A basic structural reliability problem consists of two components, the load effect \( S \) and the resistance \( R \), which constitute the limit state function, \( G = R - S \). The failure of a structure occurs when this limit state is violated. Mathematically, the probability of failure (i.e., the violation of the limit state) can be expressed as follows (Melchers, 1999):

\[
p_f = P[G = R - S \leq 0] = \iiint_D f_{SR}(s,r)dsdr \tag{Equation 2.30}
\]

In Equation 2.30, \( f_{SR}(s,r) \) is the joint density function of resistance, \( R \), and loading, \( S \). \( D \) is the domain under evaluation. This problem can be geometrically illustrated, as observed in Figure 2-21.
Figure 2-21 Geometric representation of the probability of structural failure


The fundamental method for solving Equation 2.30 is the integration method. As the name implies, the integration method works by directly performing the integration for Equation 2.30. If both the resistance (R) and the load (S) follow normal distributions, the probability of failure can be easily obtained (Cornell, 1969):

$$p_f = \Phi\left(\frac{-(\mu_R - \mu_S)}{(\sigma_S^2 + \sigma_R^2)^{1/2}}\right)$$

(Equation 2.31)

In Equation 2.31, $\mu_R$ and $\mu_S$ are mean values of $R$ and $S$, respectively. $\sigma_S^2$ and $\sigma_R^2$ are variances. However, performing direct analytic integration is only possible in some special cases. In general, the analytical solutions are difficult to be obtained if $R$ and $S$ are neither normal nor lognormal, or the limit state function is nonlinear (Melchers, 1999).

2.7.2.2 Monte Carlo Simulation

The Monte Carlo simulation is a sampling simulation technique and is the most frequently used technique in the assessments of failure (e.g., Camarinopoulos et al., 1999; Li et al., 2009; Rajani & Abdel-Akher, 2012; Sadiq et al., 2004; Yamini, 2009). The Monte Carlo simulation comprises random variable sampling from given distributions, a violation check for limit state function and a repeated violation check. The probability of structural failure can be expressed:
\[ p_f = \int \cdots \int [I(G(\mathbf{x}) \leq 0)] f_\mathbf{x}(\mathbf{x}) d\mathbf{x} \quad \text{(Equation 2.32)} \]

\[ I(G(\mathbf{x})) = \begin{cases} 1, & \text{if } G(\mathbf{x}) \leq 0 \\ 0, & \text{if } G(\mathbf{x}) > 0 \end{cases} \quad \text{(Equation 2.33)} \]

In Equations 2.32 and 2.33, \( G \) is the limit state function. \( I \) is an indicator function. If the number of trials \( (N) \) is large enough, the probability of failure can be approximated as follows:

\[ p_f \approx \frac{1}{N} \sum_{j=1}^{N} I(G(\hat{\mathbf{x}}_j) \leq 0) \quad \text{(Equation 2.34)} \]

In Equation 2.34, \( \hat{\mathbf{x}}_j \) is the \( j \)th vector of random observation from \( f_\mathbf{x}(\cdot) \). The number of trials \( (N) \) is the sample size, which affects the accuracy of the approximation.

### 2.7.2.3 First-Order Second-Moment Method

FOSM is widely used in the assessment of pipelines (e.g., Ahammed, 1998; Ahammed & Melchers, 1996; Ahammed & Melchers, 1997; Caley et al., 2002). This method begins with linearising the limit state function at the checking point on the surface point (i.e., \( G = 0 \)). This checking point is found through an iteration algorithm so that the distance between the checking point \((y_1^*, y_2^*, \ldots, y_n^*)\) and the point \((\mu_{y_1}, \mu_{y_2}, \ldots, \mu_{y_n})\) becomes the shortest. The point \((\mu_{y_1}, \mu_{y_2}, \ldots, \mu_{y_n})\) is defined by the mean values of variables. Once the checking point is determined, the mean and standard deviation of \( G \) will be determined by performing partial derivatives at this point:

\[
\mu_G = G(y_1^*, y_2^*, \ldots, y_n^*) + \left[ (\mu_{y_1} - y_1^*) \frac{\partial G}{\partial y_1} + (\mu_{y_2} - y_2^*) \frac{\partial G}{\partial y_2} + \cdots + (\mu_{y_n} - y_n^*) \frac{\partial G}{\partial y_n} \right]
\]

(Equation 2.35)

\[
\sigma_G = \left[ \left( \frac{\partial G}{\partial y_1} \right)^2 + \left( \frac{\partial G}{\partial y_2} \right)^2 + \cdots + \left( \frac{\partial G}{\partial y_n} \right)^2 \right]
\]

(Equation 2.36)

Then, the probability of failure is calculated by the following expressions:
\[ p_f = P[G \leq 0] = \phi(-\beta) \]  
(Equation 2.37)

\[ \beta = \frac{\mu_G}{\sigma_G} \]  
(Equation 2.38)

In general, this reliability method is effective in calculating the failure probability of structures if the limit state function is simple. For problems with nonlinear limit state functions, the determination of the checking point is difficult and the results of failure prediction may be inaccurate. The accuracy of this method strongly depends on the transformations of variables with non-Gaussian distributions (Caleyo et al., 2002; Melchers, 1999).

**2.7.2.4 Upcrossing Method**

In general, the loads applied to a structure fluctuate with time; the material resistance is also time-variant, due to ageing and deterioration. When the time \( t \) is considered, the assessment of structural failure becomes a time-dependent reliability problem, which can be represented as follows:

\[ p_f(t) = P[R(t) - S(t) \leq 0] = P[G(R(t), S(t)) \leq 0] \]  
(Equation 2.39)

Equation 2.39 represents a typical upcrossing problem. That is, the stochastic process \( S(t) \) crosses a threshold \( R(t) \). The failure depends on the time that is expected to elapse before the first occurrence of the upcrossing event. The failure probability of the structure is known as ‘first passage probability’ and can be determined as follows (Melchers, 1999; Ditlevsen, 1983):

\[ p_f(t) = 1 - [1 - p_f(0)]e^{-\int_0^t \nu dt} \]  
(Equation 2.40)

In Equation 2.40, \( p_f(0) \) is the probability of failure at time \( t = 0 \), \( \nu \) is the mean rate for the action process \( S(t) \) to upcross the threshold \( R(t) \). The Rice formula can be used to determine \( \nu \), as follows (Melchers, 1999):

\[ \nu = \nu_R = \frac{\infty}{\hat{R}} \hat{S}(\hat{R}) f_{S\hat{R}}(R, \hat{S}) d\hat{S} \]  
(Equation 2.41)
In Equation 2.41, $U_R^+$ is the upcrossing rate of the action process, $S(t)$, relative to the threshold process, $R(t)$; $\dot{S}$ is the slope of $S$ with respect to time ($t$); $\dot{R}$ is the time derivative process of $R$; and $f_{\dot{S}S}(\cdot)$ is the joint probability density function for $\dot{S}$ and $S$.

For a nonstationary Gaussian process, Li and Melchers (1993) derived a closed-form analytical solution to Equation 2.41, as follows:

$$U_R^+ = \frac{\sigma_{\dot{S}S}}{\sigma_S} \phi \left( \frac{R - \mu_S}{\sigma_S} \right) - \phi \left( \frac{\dot{R} - \mu_{\dot{S}S}}{\sigma_{\dot{S}S}} \right) - \Phi \left( \frac{\dot{R} - \mu_{\dot{S}S}}{\sigma_{\dot{S}S}} \right) f_{\dot{S}S}(\cdot)$$

(Equation 2.42)

This solution has been widely applied in the failure assessment of infrastructure by modelling the loading process (e.g., the applied stress and the growth of corrosion) as a nonstationary Gaussian process (e.g., Li & Mahmoodian, 2013; Li & Melchers, 2005; Li & Zhao, 2009). More recently, an analytical solution of Equation 2.41 for a nonstationary lognormal process has been derived by Li et al., (2016a):

$$U_R^+ = \frac{e^{\sigma_{\dot{S}S}(t)}}{R(t)e^\varepsilon(t)} \phi \left( \frac{\ln[R(t)] - \lambda(t)}{\varepsilon(t)} \right)$$

$$\times \left\{ \sqrt{e^{\varepsilon}(1 - \Phi \left( 1 - \frac{\ln[\dot{R}(t)] - \lambda_{\dot{S}S}(t)}{e_{\dot{S}S}(t)} \right)) - \frac{\ln[\dot{R}(t)] - e_{\dot{S}S}(t)}{\varepsilon}} \Phi \left( -\frac{\ln[\dot{R}(t)] - \lambda_{\dot{S}S}(t)}{\sigma_{\dot{S}S}} \right) \right\}$$

(Equation 2.43)

In Equation 2.43, $\varepsilon(t)$ and $\lambda(t)$ are parameters of the lognormal random variable $S(t)$ and can be determined by the following equations (Papoulis, 1965):

$$\varepsilon = \sqrt{\ln(\frac{\sigma^2}{\mu^2} + 1)}$$

(Equation 2.44)

$$\lambda = \ln \left( \frac{\mu^2}{\sqrt{\sigma^2 + \mu^2}} \right)$$

(Equation 2.45)
In summary, the fundamentals of material criteria and models that are commonly used to predict the failure loads of corroded pipes have been briefly reviewed in Section 2.7. Most of the previous research focused on the effects of corrosion, based on the strength theory rather than on fracture mechanics. Further, the corrosion pits considered in the current pipe failure models are either assumed to be blunt with a rectangular shape or simplified, by using the depth or area of corrosion point. However, corrosion pits with sharp fronts have not been sufficiently investigated and pipe failure models for sharp corrosion pits have not been developed.

The typical reliability assessment methods have been reviewed. In general, FORM is a more convenient and efficient method for most of the relatively simple reliability problems than the direct integration method. Although the Monte Carlo simulation does not need to determine the checking point and to transform the distributions of random variables, many trials (approximately $10^5$) are often required to ensure the accuracy of the approximation. Nevertheless, the Monte Carlo simulation has been successfully used in most reliability assessments because it is easy and straightforward in replicating real-world situations. Since most of the processes in practical engineering structures are time-variant, the time-dependent reliability methods, based on upcrossing theory, have important applications.

2.8 Summary

In this chapter, the current literature concerning corrosion theory, corrosion-influencing factors in soil, corrosion effect on pipe mechanical property, fracture mechanics and assessment methods have been critically reviewed. The skills of designing corrosion tests and mechanical experiments to investigate the effect of corrosion on material and mechanical properties of pipelines have been summarised. The fundamental theory of fracture mechanics and pipe failure assessments methods have been reviewed. Based on a review of the literature, there are several limits in the current assessment of buried pipe corrosion and subsequent failure assessments:

- There is a lack of large-scale laboratory corrosion tests on pipes buried in real soils and under controlled environment.
- There is a lack of research investigating the effect of corrosion on fracture toughness of pipes.
• There is a lack of research investigating the effect of sharp corrosion on the SIF of pipes.
• There is a lack of reliability assessments of corroded pipes that use more advanced upcrossing method, considering the autocorrelations of corrosion-induced mechanical degradation.
Chapter 3: Corrosion and Mechanical Tests on Pipes

3.1 Introduction

Corrosion has been well-documented as the main cause of deterioration of cast iron pipes, including wall thinning, pinhole and graphitic zone formation (Makar et al., 2001, Petersen et al., 2013). With the growth of corrosion, cast iron pipes can fail in two ways: loss of strength due to the thinning of the pipe wall and loss of toughness due to material change caused by corrosion. As reviewed in Chapter 2, most of the research conducted has investigated the effect of corrosion on strength reduction. An inspection of in-service cast iron pipe failures revealed that the most common failures of cast iron pipelines are fracture related, due to the brittle nature of cast iron material (Doyle et al. 2003, Makar et al. 2001). Despite the practical significance of this observation, it is noted that little research has been conducted to investigate the effects of corrosion on the fracture toughness of cast iron pipes. A review of the literature in Chapter 2 demonstrated that few laboratory investigations, using real soils and full-scale pipes, have been performed.

The intention of Chapter 3 is to present an experimental program, designed to investigate the effect of external corrosion of buried cast iron pipes on their mechanical properties. A new testing methodology is developed to replicate the corrosion of cast iron pipes in real soil and evaluate the effect of corrosion on degradation, in terms of fracture toughness and modulus of rupture. Further, this methodology encompasses various environments, represented by different soil pH conditions. Microstructure analyses of pipe specimens before and after corrosion were conducted to identify possible causes for degradation of mechanical properties. Specimens were compared with exhumed pipes. This research is significant because it investigates the effect of corrosion on mechanical properties—in particular, fracture toughness, which is quantitatively investigated—and the corrosion of pipe specimens by replicating actual soil conditions and therefore, representing the real-world problem. The results presented here can contribute to the body of knowledge of corrosion and its effect on the mechanical properties of buried metal pipes.
The work presented in this chapter has already been published in an international journal, *Journal of Materials in Civil Engineering* (Wang et al., 2018).

### 3.2 Experimental Methodology

#### 3.2.1 Overall Program

The proposed methodology in the current study is part of a research program to investigate the remaining life of buried ageing metal pipelines. The overall program comprises corrosion and mechanical tests on coupons, pipe sections and prototype pipe tests to estimate the realistic corrosion rates of buried metal pipes and hence, evaluate the deterioration of mechanical properties of metal pipes induced by corrosion. The current study focuses on corrosion tests on full-scale pipe sections that were buried in soil under controlled conditions of soil saturation, pH, soil density, temperature and humidity. The corrosion tests were followed by fracture toughness and crush ring tests to determine the effect of corrosion on the mechanical properties of the buried pipes. The corrosion characteristics of these pipes were thoroughly examined using various advanced techniques described in Section 3.2.

#### 3.2.2 Preparation of Corrosive Soil

The fundamental corrosion mechanism of cast iron pipes buried in soils is an electrochemical reaction, created by the interaction between iron and the corrosive agents in the ambient soil. In most soils, corrosion takes at least several years to develop to such an extent that it has a significant effect on pipe behaviour (Romanoff, 1957; Schwerdtfeger, 1953). Therefore, it is necessary to select soil with relatively high corrosiveness for laboratory corrosion tests. Generally, soils with poor aeration, high moisture content and electrical conductivity, and high levels of soluble salts are recognised as corrosive soils (Kreysa & Schütze, 2008, Romanoff, 1964). Due to its finely dispersed structure, clay soils can retain moisture more easily than other soils, such as sandy soils (Pritchard et al., 2013). Further, clay soil typically has low resistivity, due to the high content of dissolved ions in the pore water (Doyle et al., 2003). Hence, a clay soil according to Unified Soil Classification System (ASTM International, 2006) was selected in this study. As the soil was initially received as a mix of lumps, it was first crushed to small, uniform size by a crusher and then sieved using BS 410 (International Organization for Standardization, 2000) to an aperture
size of 2.36 mm before being used in the experiments. The physical and chemical properties of the clay are shown in Table 3-1 and Table 3-2, respectively.

**Table 3-1 Physical properties of soil sample**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid limit</td>
<td>29.1</td>
</tr>
<tr>
<td>Plastic limit</td>
<td>20.4</td>
</tr>
<tr>
<td>Plastic Index</td>
<td>8.71</td>
</tr>
<tr>
<td>Optimum moisture content</td>
<td>14.62 (%)</td>
</tr>
<tr>
<td>Target dry density</td>
<td>1,600 kg/m³</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>2.64</td>
</tr>
<tr>
<td>Resistivity</td>
<td>23.5 Ω m</td>
</tr>
<tr>
<td>Original pH</td>
<td>8.17</td>
</tr>
</tbody>
</table>

**Table 3-2 Chemical composition of soil sample (mg/kg)**

<table>
<thead>
<tr>
<th>Chemical</th>
<th>Content</th>
</tr>
</thead>
<tbody>
<tr>
<td>SO₄²⁻</td>
<td>50.0</td>
</tr>
<tr>
<td>NO₃⁻</td>
<td>23.1</td>
</tr>
<tr>
<td>Cl⁻</td>
<td>115.6</td>
</tr>
<tr>
<td>K⁺</td>
<td>39.1</td>
</tr>
<tr>
<td>Na⁺</td>
<td>119.0</td>
</tr>
<tr>
<td>Ca²⁺</td>
<td>333.0</td>
</tr>
<tr>
<td>Mg²⁺</td>
<td>197.0</td>
</tr>
<tr>
<td>Organic matter</td>
<td>0.26 (%)</td>
</tr>
</tbody>
</table>

To simulate and ensure the corrosiveness of soil, both the saturation and acidity in the soil were controlled. Typically, corrosion rates of metal buried in soils with some moisture are faster than metal buried in soils that are extremely dry or fully saturated (Gupta & Gupta, 1979). According to the work of Murray and Moran (1989), soil saturation of 80% could incur the most aggressive corrosion behaviour. Thus, the initial saturation of the soil was maintained at 80% (i.e., water content of 20% by weight for a target dry soil density of 1600 kg/m³).

To induce the corrosion of the pipes in soil, the acidity of the natural soil needs to be controlled. It is known that pH of soil in the realistic field usually covers a wide range, from high acidity (e.g., pH = 2.6) to high alkalinity (pH = 10.2), as documented in the NBS database (Romanoff, 1957). A recent field test on 18 sites in
Australia demonstrated that the pH of soils varies from 4.6 to 8.3 (Petersen et al., 2013). With consideration of these reported pH values in field soils, the pH levels of clay soil used in this study’s corrosion tests were adjusted to three different values (i.e., 2.5, 3.5 and 5.0) through use of diluted hydrochloric acid. The soil with pH of 2.5 was prepared to represent the most corrosive medium, while the soil with pH of 3.5 has relatively less corrosiveness. The soil with pH of 5 was selected to represent a general in situ soil condition, based on the knowledge that the corrosion rate does not rely on pH when soil pH is over 5 (e.g., Kreysa & Schütze, 2008; Nesic et al., 1996). The procedure of preparing the corrosiveness in current tests is consistent with the soil corrosiveness classification proposed by the National Soil Resources Institute in Canada (Royse et al., 2009) who determined clay content, moisture, acidity and sulphates as the most influencing factors on the corrosion of buried iron.

3.2.3 Corrosion Exposure Conditions

The corrosion exposure conditions were carefully designed and maintained to duplicate a corrosive soil environment, while reducing the variances and uncertainties caused by the spatial and temporal changes in soil properties. Two types of containers were used in the published literature (Goodman et al., 2013; Murray & Moran, 1989; Gupta & Gupta, 1979 etc.). The first option was to leave the soil container open with saturated soil, permitting no water to be supplemented during the test period. The second option was to enclose the soil container during the whole period of exposure to prevent water loss. The first approach is considered effective for short-term tests only (i.e., a few hours or a couple of days), as evaporation in a short exposure time causes relatively small water loss. For a long-term corrosion exposure, sealing the soil container prevents water loss and affects the transport of oxygen and water throughout the soil mass. As a consequence, the soil conditions and corrosion reactions on the metal surface could deviate from what occurs in field soils.

To simulate the realistic exposure condition of buried pipes, while keeping the main soil parameters (i.e., water content, resistivity and aeration) relatively constant, the soil containers in this study were not covered and the water loss induced by evaporation was compensated by periodic water additions. To facilitate this, one soil container was used for weight loss measurement by a large capacity scale. The result
was applied to other containers to determine the water supplementing required. Two moisture sensors (MP 406 dielectric theory-based) were buried at the pipe burial level in each of the soil containers to monitor the moisture of soils. All tests were conducted in an environmental chamber (see Figure 3-1), in which temperature and humidity were maintained at 24 °C and 50% RH, respectively.

![Environmental chamber](image)

**Figure 3-1 Environmental chamber**

The burial depth of pipe sections in this study was 300 mm, based on field observations and reported literature (e.g., Goodman et al., 2013; Petersen & Melchers, 2012). For example, in Australia, most of the water pipes were buried with less than 1 m cover heights, as was demonstrated by a recent field inspection conducted by Petersen and Melchers (2012).

### 3.2.4 Test Types

A series of tests was conducted in the current study to examine corrosion behaviour and investigate the influence of corrosion on the mechanical properties of the pipe. These tests included corrosion current and mass loss measurements, fracture toughness tests, and pipe crush ring and load capacity tests. Two modes of failure have been considered, namely wall rupture due to strength loss and fracture due to the stress concentration at the tips of corrosion pits (Li & Yang, 2012; Rajani, 2000; Wang et al., 2017). As discussed in Section 3.2.2, three soil containers with soil pH levels of 2.5, 3.5 and 5.0 were designed to investigate the effect of corrosion on the
fracture toughness of pipe after different corrosion exposure. A schematic of tests and exposure time of specimens is shown in Figure 3-2. Three pipe sections were buried in one soil container. Mass loss measurements were taken on one corroded pipe section from each of the containers after 210, 365 and 540 days of exposure, according to ASTM International (2011). After taking the corrosion measurements, four duplicate specimens were machined off from each pipe section for fracture toughness tests. Additionally, a container with soil pH of 2.5 was established and ring tests were conducted on each pipe section after corrosion exposure time of 210, 365 and 540 days to investigate the effect of corrosion on the modulus of rupture of the pipe over time. The test variables considered in the current study are summarised in Table 3-3.
### Table 3-3 Test variables in pipe section tests

<table>
<thead>
<tr>
<th>Mechanical Tests</th>
<th>Corrosion Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Exposure Period (Days)</td>
</tr>
<tr>
<td>Fracture toughness</td>
<td>0</td>
</tr>
<tr>
<td>Ring test</td>
<td>0</td>
</tr>
</tbody>
</table>

### 3.3 Design of Specimens

#### 3.3.1 Selection of Materials

The material used in this study is grey cast iron, due to its widespread application in the water, oil and gas transmission sector. To manufacture pipe specimens with realistic material composition representing those of in-service pipelines, the element composition tests were conducted (see Table 3-4) on two exhumed cast iron pipes obtained from local water utilities. The comparison of the elemental composition with current Australian specifications for cast iron material shows that T220 grey cast iron has a similar composition to the exhumed pipe material compositions. Therefore, T220 grey cast iron was selected in this study to make pipe specimens.

### Table 3-4 Chemical composition of cast iron

<table>
<thead>
<tr>
<th>Samples</th>
<th>C</th>
<th>Si</th>
<th>Mn</th>
<th>Mg</th>
<th>Al</th>
<th>P</th>
<th>S</th>
</tr>
</thead>
<tbody>
<tr>
<td>New pipe specimens</td>
<td>3.380</td>
<td>2.220</td>
<td>0.750</td>
<td>0.009</td>
<td>0.006</td>
<td>0.007</td>
<td></td>
</tr>
<tr>
<td>Exhumed pipe A</td>
<td>3.58</td>
<td>2.48</td>
<td>0.74</td>
<td>&lt; 0.01</td>
<td>&lt; 0.01</td>
<td>0.67</td>
<td>0.06</td>
</tr>
<tr>
<td>Exhumed pipe B</td>
<td>3.47</td>
<td>2.29</td>
<td>0.41</td>
<td>&lt; 0.01</td>
<td>&lt; 0.01</td>
<td>0.84</td>
<td>0.08</td>
</tr>
</tbody>
</table>

#### 3.3.2 Specimens for Corrosion and Ring Tests

The pipe specimens were cast in sand moulds with the same dimensions as that of the exhumed old pipes (i.e., outer diameter of 120 mm and thickness of 11 mm) from a local casting factory. A vertical cast approach was used to ensure the uniform wall thickness. All pipes were cast in 1 m lengths; smaller sections (120 mm) were cut for corrosion tests and subsequent mechanical tests. Each small pipe section’s interior was coated with epoxy and the ends were sealed by polyethylene caps. The epoxy and polyethylene are anti-corrosive materials and hence, are used to prevent the accession of air and seepage of moisture from soil into the interior of the pipe.
copper wire was connected to each specimen to monitor the electrochemical parameters (i.e., corrosion potential and current).

### 3.3.3 Specimens for Fracture Toughness Test

In the current study, the fracture toughness of corroded pipes was determined using single-edge bend specimens that were cut from the pipe wall, as illustrated in Figure 3-3. Two types of specimens were used in the fracture toughness tests. One was a standard specimen with a square cross-section, as recommended by ASTM E399 (ASTM International, 2012a). Since the external surface of the pipe wall was flattened to produce this type of specimens, they are referred to as machined specimens in this work. The machined specimens were prepared from both corroded and non-corroded pipes. Four duplicates were made from each machined pipe section to allow for the effects of material and test system variability. To consider the effect of external corrosion on the fracture toughness of pipe, another type of specimen was made with the external surface of pipe unmachined (called unmachined specimens). Four duplicates were cut from each unmachined pipe specimen. The total number of specimens used in fracture toughness test was 56. The dimensions of specimens were predetermined by the exhumed pipes, which were 10 mm thick, 20 mm wide and 120 mm long. The pre-crack was introduced to the specimens by machining a narrow notch, 9 mm in length, followed by fatigue cracking ($10^4 \sim 10^6$ loading cycles) to reproduce the sharpest possible crack. The fatigue crack extension was controlled between 1.3 mm and 2.0 mm.

Depending on the width of the specimens, there can be two types of fractures: plane stress fracture and plane strain fracture. For the plane stress fracture, the fracture toughness decreases with the increase of specimen width and stabilises at a certain width for which plane strain fracture occurs. This width is determined according to ASTM E399-12 (ASTM International, 2012a) as follows:

$$B \geq 2.5 \left( \frac{K_{IC}}{\sigma_{YS}} \right)^2$$  \hspace{1cm} (Equation 3.1)

In Equation 3.1, $B$ is the width of the specimen, $K_{IC}$ is the plane strain fracture toughness and $\sigma_{YS}$ is the 0.2% offset yield strength of the material.
Figure 3-3 Fracture toughness specimens cut from pipe section

Note: Unit of measurement in mm.

It is noted that, for the grade of cast iron material used in these tests, the width of the specimen calculated by Equation 3.1 is 20 mm, which is larger than most in-service pipelines. Since the purpose of this study is to examine how corrosion affects the fracture toughness of cast iron pipes of a given size over time, but not determine accurately the absolute value of plane strain fracture toughness, it is justifiable to select a smaller size, based on the real wall thickness of cast iron pipelines. Importantly, the thickness of exhumed pipes used for comparison is 10 mm. Nevertheless, as differentiation, the fracture toughness determined in this work is quoted as a provisional fracture toughness value, $K_Q$.

3.4 Test Setup and Procedure

In each soil container, the pipe specimens were buried end to end with 60 mm distance between each specimen to reduce the influence of galvanic action between specimens. Both the bedding height and the distance between the pipe surface and container wall were designed to be the same as the burial depth (i.e., 300 mm). A schematic of the soil–pipe assembly is shown in Figure 3-4. To have uniform soil density, clay soil was compacted in layers of 50 mm in height. The uniform compaction was ensured by wet tamping, using a medium scale drop hammer (with a weight of 3.5 kg, performing 180 blows per layer). In each container, two moisture sensors, three thermocouples and two pH electrodes were buried at the pipe burial
level. All sensors were connected to a data logger, which recorded the measured data every hour. One soil container was placed on the high capacity digital scale to measure the weight loss of the container each week. Based on the weight loss, the water was supplemented. During the corrosion exposure, corrosion current was measured by use of an ACM LPR in situ instrument, which provided an instantaneous measurement of the corrosion rate for pipe sections. After 210 days, 365 days and 540 days, the mass loss was measured on each excavated specimen, according to ASTM G1-03 (ASTM International, 2011).

Figure 3-4 Typical soil–pipe assembly and corrosion test setup
3.5 Measurement of Corrosion and its Effect

3.5.1 Corrosion Current and Mass Loss

Corrosion current in the real system cannot be measured directly. Instead, it can be estimated by using electrochemical techniques, such as the LPR method. The electrochemical corrosion monitoring of the experimental setup was achieved with an LPR instrument. The measurement was performed on a three-electrode system, which included a copper–copper sulphate reference electrode, a platinum auxiliary electrode and a working electrode of the metal under test. The potential step approach was used with the start potential of 10 mV and the stop potential of –10 mV. Since LPR is a well-established method and the focus of this research is not on the LPR method, the details of the method are not repeated in this thesis, but can be sourced in ASTM International (2004) and Millard et al. (2001). The corrosion current density was calculated as follows (ASTM International, 2004):

\[ i_{\text{corr}} = \frac{B}{R_p} \]  
(Equation 3.2)

\[ B = \frac{\beta_A \beta_C}{2.303 (\beta_A + \beta_C)} \]  
(Equation 3.3)

In Equations 3.2 and 3.3, \( R_p \) is polarisation resistance in ohm-cm\(^2\), \( B \) is the Stern-Geary constant. This is normally assumed to be 26 mV for the active corrosion state (Andrade & Alonso, 1996). That is, the Tafel slopes \( \beta_C \) and \( \beta_A = 120 \text{ mV/decade} \). \( R_p \) was directly measured from the instrument. It is assumed that the corrosion distributes uniformly on the exposed surface. The resistance of soil was subtracted from the resistance of the whole system by IR compensation technique. This was performed by applying a high frequency (300 Hz) voltage input signal (31 mV amplitude) between the auxiliary electrode and the working electrode.

Mass loss measurement was performed on corroded pipe specimens at designated points of time (i.e., 210, 365 and 540 days) as per ASTM G1-03 (ASTM International, 2011). After the corroded specimens had been cleaned by a solution comprising 2,000 ml of 38% HCl, 100 g stannous chloride and 40 g antimony...
trioxide, the mass loss was calculated as a reduction of pipe weight before and after corrosion, expressed in g/cm².

3.5.2 Fracture Toughness

In the current test, three-point bending tests were first conducted to load fatigue pre-cracked specimens to induce unstable crack extension, resulting in a single point value for fracture toughness at the point of fracture instability. The specimens were then loaded such that the rate of increase in the stress intensity factor ranged from 0.55 to 2.75 N/m³/² with a support span of 80 mm (i.e., four times the specimen width). A continuous measurement of CMOD versus force was attained by a clip gauge that was attached to specimens across the notch mouth, as shown in Figure 3-5a. Based on the plot of force-clip gauge displacement, known as p-v curve, the Mode I fracture toughness can be determined by the following equations (ASTM International, 2012a):

\[
K_I = \frac{P_Q S}{B W^{3/2}} f\left(\frac{a}{W}\right)
\]  
(Equation 3.4)

\[
f\left(\frac{a}{W}\right) = \frac{3\left(\frac{a}{W}\right)^{1/2} \left[1.99 - \left(\frac{a}{W}\right)(1-\frac{a}{W})(2.15 - 3.93(\frac{a}{W}) + 2.7(\frac{a}{W})^2)\right]}{2\left(1 + 2\frac{a}{W}\right)(1-\frac{a}{W})^{3/2}}
\]  
(Equation 3.5)

In Equations 3.4 and 3.5, \(P_Q\) is the conditional load at fracture (which is determined by drawing the secant line through the origin of the p-v curve, with a slope equal to 95% of the initial linear portion of the curve), \(S\) is the support span, \(B\) is specimen thickness, \(W\) is specimen width and \(a\) is crack size (which equals to notch length plus the fatigue crack extension).
3.5.3 Modulus of Rupture

It is well-known that stresses due to external loads on cast iron pipes are usually not directly considered (Moser & Folkman, 2001). Instead, modulus of rupture is often determined and acts as a basic parameter that characterises the loading capacity of the pipe against the external loading (e.g., the vertical earth load above the pipe and traffic loads; Seica & Packer, 2004). The modulus of rupture is commonly determined in the laboratory by crush ring tests with either a parallel plate or a three-edge loading condition, and can be expressed as follows (Seica & Packer, 2004):

\[
\sigma = 954 \frac{P_{\text{max}}(I + \bar{t})}{L \cdot \bar{t}^2}
\]

(Equation 3.6)

In Equation 3.6, \( \sigma \) is the modulus of rupture (in MPa), \( P_{\text{max}} \) is the load applied at fracture (in N), \( L \) is the mean length of pipe section (in mm), \( I \) is the mean internal diameter of the ring (in mm) and \( \bar{t} \) is the mean wall thickness of the ring measured at the fracture position (in mm).

In this study, the ring crush test was conducted on pipe sections of 120 mm in length and crushed between parallel plates using an MTS machine of 1,000 KN capacity.
ensure good contact between loading plate and ring specimens, a resin cushion of approximately 1 mm thick was laid on the bed and bottom of the loading plates of the machine. The ring was put in place so that the bottom element rested on the cushion. The head of the machine was then pressed down at the rate of 0.3 mm per minute until failure occurred. At each crack appearing, its size and location were carefully noted. This was continued until the pipe reached complete rupture (i.e., failure). After the tests, the wall thickness at fracture was measured by a digital calliper to the nearest 0.01 mm.

3.6 Results and Analysis

3.6.1 Corrosion Current Density

Since it takes a long time to produce data, the results presented here are only for specimens buried in soils of pH 2.5, 3.5 and 5.0 up to 365 days. Results of the corrosion current density (mA/cm²) of pipe specimens buried in the soil of pH 2.5, 3.5 and 5.0 with an exposure period of 365 days are presented in Figure 3-6, in which each plotted point represents an average of three testing results. The mean of the standard deviations of measurements in duplicates is simplified as ‘mean SD’. It can be observed that although the current density of specimens is largely scattered due to the random occurrence and growth of corrosion in the soil, the trend of current density for specimens in various soil pH is clear. It can be noted from Figure 3-6 that pipe specimens experienced relatively high current densities at the beginning of the corrosion test (e.g., $i_{corr} > 0.02$ mA/cm² for pH of 2.5) and the current densities gradually decreased with longer exposure time (e.g., $i_{corr} < 0.01$ mA/cm² for pH 3.5 and pH 5.0 after 250 days). This indicates that the corrosion rate is high at the initial exposure stage; however, it reduces and stabilises by a small value over time. This is not unexpected; in the early stage of corrosion, the pipe surface was completely exposed to a high concentration of hydrogen, iron and oxygen, which caused a high corrosion rate. At longer terms, an adherent layer of corrosion products formed a protective barrier against corrosion (see Figure 3-7) and consequently, the presence of a rust (oxide) layer slowed or prevented the transportation of reactants (H⁺ or O₂) to the steel substrate, resulting in a decreased corrosion rate. These results are consistent with other results reported in the literature (Hou et al., 2016; Mohebbi & Li, 2011; Schwerdtfeger, 1953). The decreased corrosion rate was also associated
with the diffusion processes of corrosion reactants through the soil matrix, because the supply of corrosive agents (e.g., acidity and oxygen in soil) is not readily available as in aqueous solution.

Figure 3-6 Corrosion current density of pipes buried in soils with (a) pH 2.5, (b) pH 3.5 and (c) pH 5.0

Corrosion current densities in soils with pH 2.5 and 5.0 have the largest and the smallest values respectively, showing that current densities are larger for more acidic soils. For example, the current density of pipe specimens in soil with pH of 2.5 is higher than that in soil with a pH of 3.5 by about 50%. This is a direct result of the fact that soil with smaller pH has a higher concentration of hydrogen ions. This also can be observed in Figure 3-7; a thicker adherent dark layer, consisting of graphite and corrosion products, formed on the surface of the pipe specimens exposed to soil with pH of 2.5.
The corrosion potentials of pipes buried in different soils have been measured with respect to the reference electrode and the results are presented in Figure 3-8. Generally, pipes buried in soils with a lower pH have more negative corrosion potentials. For example, the averaged corrosion potentials in pipes buried in soils with pH of 2.5, 3.5 and 5.0 are $-732 \text{ mV}$, $-584 \text{ mV}$ and $-504 \text{ mV}$ with respect to Cu/CuSO$_4$ electrode, respectively. This means that the corrosion reaction is more thermodynamically active in soil with lower pH. Additionally, it can be observed that the corrosion potentials in all pipes moved towards more positive potential over time; however, the corrosion potentials of pipes in soil with a pH level of 2.5 did not increase until approximately 200 days of exposure.
3.6.2 Mass Loss

The mass loss results of specimens exposed to soil with pH of 2.5, 3.5 and 5.0 for exposure of 365 days are calculated and converted to mils per year (mpy), as shown in Figure 3-9. The density of material is assumed to be 7.2 g/cm³, according to ASTM G1-03 (ASTM International, 2011). As expected, although more metal loss was caused with longer exposure times and lower pH levels, the corrosion rates generally decreased in all cases. For example, in specimens buried in soil with pH of 2.5, the corrosion rate was 16.56 mpy after 7 months of exposure and 12.46 mpy at the end of 12 months. In comparison, the pipe specimens buried in soil with pH 3.5 and 5.0 experienced less corrosion. The slopes of corrosion rates in both pH 2.5 and 3.5 indicated a reduction in corrosion rates over exposure time. A variance of mass loss results was also expected, due to the stochastic nature of corrosion and measurement errors. The measurement errors of mass loss can occur during the process of weighing before and after tests and cleaning specimens; however, in theory this kind of error should not happen. Overall, this result is in reasonable agreement with the preceding corrosion current measurements and those in published reports (e.g., Murray & Moran, 1989; Romanoff, 1957).
3.6.3 Fracture Toughness Reduction

The results of the fracture toughness test of pipe specimens are summarised in Table 3-5. Each specimen is labelled by its exposure environment: pH-exposure time-duplicate number. For example, specimen 2.5-365-1 represents the pipe section buried in soil with pH 2.5 for 365 days as duplicate 1. The suffix ‘m’ in name of specimens in the first three rows represents a machined specimen. The rest of specimens are unmachined. The results contain loading measurements at the point of unstable fracture occurring on specimens exposed to soil with pH 2.5, 3.5 and 5.0 up to 365 days’ corrosion exposure. A comparison of results between machined and unmachined specimens shows that a higher fracture toughness has been produced by unmachined specimens due to the additional curved part of material.

A clear variation in results can be observed between the duplicate specimens. This is mainly attributed to the variation of corrosion, material and geometric properties of specimens and in part, due to minor measurement errors that resulted from either pre-crack fatigue or three-point bending tests. Despite the observed variation, a plot of the averaged fracture toughness of duplicates (see Figure 3-10) indicates a decreasing trend of fracture toughness with longer exposure time for specimens buried in soil with various pH levels. To be specific, the reduction of fracture toughness after 365 days corrosion exposure for pH 5.0, 3.5 and 2.5 are 6.22%, 6.70% and 20.29%, respectively. As anticipated, the reduction of fracture toughness of specimens exposed to soil with pH of 2.5 is larger than that of specimens in the soil of pH 3.5.
and 5.0. Additionally, the specimens in soil with a pH of 2.5 had a larger variation rate of corrosion current density (i.e., the slope of curve) than specimens in soil with a pH of 3.5 and 5.0.

Table 3-5 Summary of the results of the fracture toughness test

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>K&lt;sub&gt;0&lt;/sub&gt;</th>
<th>a&lt;sub&gt;0&lt;/sub&gt;</th>
<th>W</th>
<th>B</th>
<th>a/W</th>
<th>P&lt;sub&gt;max&lt;/sub&gt;</th>
<th>P·Q</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MPa·m&lt;sup&gt;0.5&lt;/sup&gt;</td>
<td>Unit (mm)</td>
<td>Unit (N)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>initial-1m</td>
<td>14.72</td>
<td>1.68</td>
<td>20.04</td>
<td>9.95</td>
<td>0.533</td>
<td>2106.29</td>
<td>1750.00</td>
</tr>
<tr>
<td>initial-2m</td>
<td>13.81</td>
<td>1.57</td>
<td>19.89</td>
<td>9.88</td>
<td>0.518</td>
<td>2032.41</td>
<td>1620.00</td>
</tr>
<tr>
<td>initial-3m</td>
<td>13.80</td>
<td>1.77</td>
<td>19.99</td>
<td>9.82</td>
<td>0.539</td>
<td>1979.63</td>
<td>1583.00</td>
</tr>
<tr>
<td>initial-1</td>
<td>16.87</td>
<td>1.85</td>
<td>19.87</td>
<td>10.17</td>
<td>0.546</td>
<td>2429.68</td>
<td>1937.50</td>
</tr>
<tr>
<td>initial-2</td>
<td>16.84</td>
<td>1.50</td>
<td>19.85</td>
<td>10.07</td>
<td>0.529</td>
<td>2585.38</td>
<td>2025.00</td>
</tr>
<tr>
<td>initial-3</td>
<td>14.84</td>
<td>1.89</td>
<td>19.61</td>
<td>10.22</td>
<td>0.555</td>
<td>2020.40</td>
<td>1625.00</td>
</tr>
<tr>
<td>initial-4</td>
<td>15.75</td>
<td>2.27</td>
<td>19.68</td>
<td>10.02</td>
<td>0.573</td>
<td>2120.31</td>
<td>1600.00</td>
</tr>
<tr>
<td>2.5-210-1</td>
<td>11.06</td>
<td>1.42</td>
<td>20.06</td>
<td>9.45</td>
<td>0.519</td>
<td>2001.15</td>
<td>1310.00</td>
</tr>
<tr>
<td>2.5-210-2</td>
<td>15.62</td>
<td>2.37</td>
<td>19.86</td>
<td>9.66</td>
<td>0.573</td>
<td>1979.00</td>
<td>1550.00</td>
</tr>
<tr>
<td>2.5-210-3</td>
<td>16.53</td>
<td>1.32</td>
<td>19.87</td>
<td>9.18</td>
<td>0.519</td>
<td>2334.39</td>
<td>1875.00</td>
</tr>
<tr>
<td>2.5-365-1</td>
<td>14.27</td>
<td>1.42</td>
<td>19.92</td>
<td>10.02</td>
<td>0.523</td>
<td>2708.85</td>
<td>1751.13</td>
</tr>
<tr>
<td>2.5-365-2</td>
<td>12.10</td>
<td>1.70</td>
<td>19.96</td>
<td>10.41</td>
<td>0.536</td>
<td>2166.55</td>
<td>1480.36</td>
</tr>
<tr>
<td>2.5-365-3</td>
<td>12.32</td>
<td>1.42</td>
<td>20.03</td>
<td>10.25</td>
<td>0.520</td>
<td>2260.36</td>
<td>1600.00</td>
</tr>
<tr>
<td>3.5-210-1</td>
<td>15.98</td>
<td>1.58</td>
<td>20.05</td>
<td>9.44</td>
<td>0.528</td>
<td>2635.41</td>
<td>1837.50</td>
</tr>
<tr>
<td>3.5-210-2</td>
<td>15.09</td>
<td>1.79</td>
<td>19.90</td>
<td>9.53</td>
<td>0.542</td>
<td>2743.92</td>
<td>1650.00</td>
</tr>
<tr>
<td>3.5-210-3</td>
<td>16.03</td>
<td>1.33</td>
<td>19.88</td>
<td>9.72</td>
<td>0.520</td>
<td>3363.25</td>
<td>1925.00</td>
</tr>
<tr>
<td>3.5-210-4</td>
<td>21.79</td>
<td>1.50</td>
<td>19.88</td>
<td>9.53</td>
<td>0.528</td>
<td>3668.66</td>
<td>2550.00</td>
</tr>
<tr>
<td>3.5-365-1</td>
<td>17.36</td>
<td>1.66</td>
<td>20.03</td>
<td>9.50</td>
<td>0.532</td>
<td>2870.98</td>
<td>1975.00</td>
</tr>
<tr>
<td>3.5-365-2</td>
<td>12.99</td>
<td>1.40</td>
<td>19.92</td>
<td>9.71</td>
<td>0.522</td>
<td>3199.51</td>
<td>1550.00</td>
</tr>
<tr>
<td>3.5-365-3</td>
<td>14.95</td>
<td>1.57</td>
<td>19.89</td>
<td>10.00</td>
<td>0.532</td>
<td>3538.20</td>
<td>1775.00</td>
</tr>
<tr>
<td>5.0-210-1</td>
<td>13.60</td>
<td>1.77</td>
<td>20.04</td>
<td>10.82</td>
<td>0.538</td>
<td>2774.94</td>
<td>1779.63</td>
</tr>
<tr>
<td>5.0-210-2</td>
<td>15.85</td>
<td>2.17</td>
<td>20.04</td>
<td>8.88</td>
<td>0.558</td>
<td>1834.09</td>
<td>1546.99</td>
</tr>
<tr>
<td>5.0-210-3</td>
<td>16.85</td>
<td>1.76</td>
<td>19.99</td>
<td>9.24</td>
<td>0.538</td>
<td>3223.77</td>
<td>1823.91</td>
</tr>
<tr>
<td>5.0-365-1</td>
<td>15.69</td>
<td>1.50</td>
<td>19.38</td>
<td>9.58</td>
<td>0.542</td>
<td>2971.78</td>
<td>1658.72</td>
</tr>
<tr>
<td>5.0-365-2</td>
<td>14.61</td>
<td>1.50</td>
<td>19.99</td>
<td>9.94</td>
<td>0.525</td>
<td>2827.30</td>
<td>1774.39</td>
</tr>
<tr>
<td>5.0-365-3</td>
<td>15.22</td>
<td>1.41</td>
<td>20.13</td>
<td>9.53</td>
<td>0.517</td>
<td>3323.75</td>
<td>1838.42</td>
</tr>
</tbody>
</table>
3.6.4 Modulus of Rupture Reduction

The results from the ring crush tests are presented in Figure 3-11 for pipe sections at the exposures of 0, 210 and 365 days. A crack has been observed to appear either at the top or bottom of the pipe at a load close to maximum ring strength. An increase of load causes this crack to extend immediately throughout the wall of pipe and break (i.e., rupture). This occurred at the top or bottom for all pipe specimens. Results revealed a slight reduction of crushing strength due to corrosion. On average, the results showed a reduction of 3.7% and 6.7% in ring crush strength in the cast iron pipe sections after corrosion exposure of 210 days and 365 days, respectively. These results were from specimens in pH 2.5 soil. In comparison, it can be observed from Figure 3-10 and Figure 3-11 that the reduction of modulus of rupture, due to corrosion, is smaller than that of fracture toughness for specimens in soil with a pH of 2.5 with same exposure time.
3.7 Observation and Discussion

From the test results, further observation and discussion can be made. Although the electrochemical techniques, such as LPR, provides a broad indication of the corrosion rate for different soil conditions, approximations were made for certain parameters for corrosion evaluation. For example, in the current study, the values of cathodic and anodic Tafel slopes (i.e., $\beta_C$ and $\beta_A$) were assumed to be the same and the static values were the same as those in most reported literature (e.g., Andrade & Alonso, 1996; Millard et al., 2001). However, the anodic and cathodic polarisation mechanisms are not identical and some dynamic processes, such as mass transfer effect and corrosion product movement, cannot be represented by these two values (Marcus, 2011). According to Andrade and Alonso (1996), $B$ varies only between 13 mV and 52 mV for most of metal/electrolyte systems. A value of 26 mV was found appropriate for the active state and 52 mV for the passive state. Nevertheless, it can be observed from Figure 3-6 that the trend of current densities for specimens in soil with various pH is clear. That is, the corrosion rate is relatively high at the early stage of the corrosion process and decreases over time.

Figure 3-6 also indicates that the corrosion measurement is very scattered and the corrosion rates (i.e., the slope of the curves) are irregular due to the randomness associated with corrosion occurrence and growth in soil. Despite this, Figure 3-6 shows that when the exposure time increased, the difference of current densities between different pH decreased. This means that, although a smaller pH can initiate
more rapid growth of corrosion at the early period of exposure, the contribution from the H⁺ reduction to the overall cathodic reaction decreased with longer time due to the consumption of acidity in the exposure medium and diffusion processes. The monitoring result of soil pH in different containers is presented in Figure 3-12, from which it can be observed that soils with initial pH values of 2.5 and 3.5 had relatively stable pH values over the exposure time; however, there is slight increase of pH over time in soil with the target pH value of 5.0. Further, Figure 3-6 demonstrates that the difference in current densities between pipes in soils with pH of 3.5 and 5.0 is not 10 times, as expected because there is over 10 times the concentration of hydrogen ions. This suggests that, although H⁺ reduction was the dominant cathodic reaction in the acidic soil, other cathodic reactions (such as O₂ or H₂CO₃ reduction) could have taken place.

![Figure 3-12 Measurements of pH in different soils](image)

In soil with pH below 5.0, corrosion mainly occurs by the liberation of hydrogen (Rossum, 1969) and the cathodic reaction can be expressed as follows (Marcus, 2011):

\[
2H^+ + 2e^- \rightarrow H_2
\]  
(Equation 3.7)

In comparison, in soil with pH of 5.0, a relatively weak H⁺ reduction reaction is expected and the oxygen reduction can become the dominant cathodic reaction, considering the abundant oxygen supply in soil with a saturation of 80%. The corresponding cathodic reaction can be expressed as follows (Marcus, 2011):
\[ O_2 + 4H^+ + 4e^- \rightarrow 2H_2O \]  \hspace{1cm} \text{(Equation 3.8)}

It is also worth noting that, due to the presence of abundant CO\textsubscript{2} in soil, a weak acid (H\textsubscript{2}CO\textsubscript{3}) is often formed when it is hydrated. According to Nesic et al. (1996), at moderate pH values (i.e., 4.0 to 6.0), the direct reduction of H\textsubscript{2}CO\textsubscript{3} also becomes important to make soil corrosive, in addition to the reduction of H\textsuperscript{+} (see Equation 3.7). The corrosion reaction can be represented as follows (e.g., Nesic et al., 1996):

\[ 2H_2CO_3 + 2e^- \rightarrow H_2 + 2HCO_3^- \]  \hspace{1cm} \text{(Equation 3.9)}

To correlate the basic electrochemical theories with the actual corrosion of buried pipe, XRD tests were conducted on powdered rust that was removed from corroded pipe specimens and the category of corrosion products was identified. The typical results of XRD are presented in Figure 3-13. It has been found that the surface corrosion products were similar for all pipe sections in different exposure environments. The corrosion products always present in the rust layers are goethite (Fe\textsubscript{3}O(OH)), hematite (Fe\textsubscript{2}O\textsubscript{3}), magnetite (Fe\textsubscript{3}O\textsubscript{4}), iron sulphite (FeSO\textsubscript{3}) and some iron oxide complex, such as iron phosphate hydroxide hydrate and whitmoreite. Interestingly, it has been found that iron carbonate (FeCO\textsubscript{3}) was found in rust samples removed from the pipe specimens exposed to soil with pH of 3.5 and 5.0. This finding confirmed that corrosion in different conditions has different forms. The reduction of H\textsubscript{2}CO\textsubscript{3} (see Equation 3.9) also occurred in soil with less acidity (i.e., soil with pH of 3.5 and 5.0). Ideally, all corrosion products can be quantified. This has been attempted in this study, but it was found very difficult to produce an accurate result due to the lack of crystalline phases for corrosion products and the constraints of the XRD technique. For example, some water-soluble chemicals (such as FeCl\textsubscript{2} and FeSO\textsubscript{4}) are expected to be present, but they cannot be detected by XRD.
Figure 3-13 Typical X-ray diffraction results for (a) pH 2.5, (b) pH 3.5 and (c) pH 5.0
The surfaces of pipes were examined by Raman spectroscopy. The Raman instrument, used to collect the spectra, was a XploRA Horiba confocal Raman probe system. The fibre optic diode laser provided excitation at 532 nm with 50 mW power at the source. Confocal objective lens of 50X was employed and the spectra were collected for 1 s for 10 ~ 30 accumulations. The results of the Raman spectra for pipes buried in different soils are presented in Figure 3-14. The results indicated that the rusts on pipes buried in all conditions have a mixture of hematite and goethite, with representative bands at 214, 270 and 396 cm\(^{-1}\) as reported in De la Fuente et al. (2016). Strong peaks for hematite appeared in XRD results (see Figure 3-13b and c). It was found that feroxyhyte with the typical bands at 396 and 685 cm\(^{-1}\) may be formed in the pipes buried in soil of pH 3.5. In addition, the comparison of Figure 3-14c with the results reported by Saheb et al. (2011), shows that siderite exists in pipes buried in soil of pH 5.0, which again confirmed the XRD results.

Pipe surfaces were cleaned to examine the corrosion-induced damage. The photographs of corroded pipe sections are shown in Figure 3-15. The inspection revealed that in more acidic soil (e.g., pH of 2.5 and 3.5), the pipe surface that contacted with acidic soil lumps was preferentially damaged; consequently, closely distributed and relatively blunt pits occurred (see Figure 3-15). This can explained by the reduction of hydrogen ions as the dominant cathodic reaction, meaning that corrosion mainly occurred in the form of hydrogen liberation (see Equation 3.7). Conversely, the corrosion pits formed on the specimen corroded in soil with pH of 5.0 are widely distributed (i.e., sparse) and deeper than the pits shown in Figure 3-15c. With the less cathodic reaction in the form of liberation of hydrogen, localised pitting corrosion is more likely to occur, due to the formation of oxygen concentration cells on the metal surface (Romanoff, 1964). The oxygen concentration cells are often caused by local variations in the oxygen and moisture content of the soil (Romanoff, 1964) and the permeability of the soil to oxygen and moisture at different locations (i.e., bottom and top of the buried pipe; Fitzgerald, 1968).
Figure 3-14 Typical Raman spectra of rust formed on pipes buried in soil for (a) pH 2.5, (b) pH 3.5 and (c) pH 5.0
In addition, inspection of two exhumed in-service old grey cast iron pipes (over 50 years old) obtained from local sites (see Figure 3-15d and 3-15e) showed that there was a clear difference in terms of pit depth between the old pipes and pipe specimens corroded in the corrosion tests. However, there were similar pit patterns. It seems reasonable to postulate that the corrosion results obtained from well-controlled...
laboratory tests can be used to predict the realistic corrosion behaviour of buried pipes, if proper control can be achieved regarding soil conditions.

It has also been observed in Figures 3-10 and 3-11 that the reduction of mechanical properties, both fracture toughness and modulus of rupture, generally follow the same trend as corrosion current density and mass loss. It shows that, with longer corrosion exposure, more reduction of mechanical properties for both fracture toughness and rupture is induced. To be specific, the degree of reduction for specimens buried in soil with pH 3.5 and 5.0 is quite small (i.e., the reductions of fracture toughness are less than 6.8% for specimens corroded in soil with pH 3.5 and that in soil with pH 5.0 after 365 days). In comparison, the pipe corroded in soil with a pH of 2.5 experienced larger reduction of fracture toughness and the plot of reduction is almost a straight line, as shown in Figure 3-10. This implies that corrosion in soil with a pH of 2.5 can accelerate the degradation of mechanical properties (e.g., fracture toughness) more easily.

A plot of fracture toughness reduction with mass loss is shown in Figure 3-16. In Figure 3-16, although a larger scatter was observed for fracture toughness reduction with respect to mass loss than that with respect to corrosion exposure time, a lower pH value shows a clear trend. In general, the reduction of fracture toughness is approximately linear with mass loss, exhibiting that mass loss can be used as a parameter to indirectly represent the degree of fracture toughness deterioration. For given results, linear relation yielded the best $R^2$, so linear fitting was used. The fitted polynomials represent for the trends that how fracture toughness reduction changes against mass loss. Since the intercept are negative at two cases, the estimation of fracture toughness reduction at mass loss in zero is meaningless. In comparison, the reduction of modulus of rupture is smaller with respect to mass loss than fracture toughness, as shown in Figure 3-11.
To further verify the degraded mechanical behaviour of corroded pipe sections, SEM tests were conducted. The specimens with dimensions of 10 mm × 10 mm × pipe wall thickness were machined from pipes. These samples were mounted in conductive Bakelite and then subjected to metallographic preparation, resulting in a 0.1 μm surface finish. The results of pipe exposed to pH 2.5 and 3.5 for 365 days are illustratively presented in Figure 3-17. It can be observed from Figure 3-17a that the morphology of cast iron material with no corrosion is typically characterised by the presence of graphite flakes (i.e., the long black plates) in the matrix of iron; however, from Figure 3-17b and 3-17c, localised corrosion and graphitisations zones are the primary forms of deterioration for cast iron. The change of morphology, as shown in Figure 3-17b and 3-17c, indicates that besides the corrosion-induced pits at the top surface of the external wall, corrosion penetrated the substrate of cast iron through the graphite flakes, causing degrading of their inherent properties. It is known that the presence of graphite flakes in cast iron can generate microcracks on the surface of corrosion pits (Conlin & Baker, 1991). Since these microcracks can allow easy access of corrosion reactants from soil medium to the substrate, the corrosion process will be accelerated. This type of microcrack was also observed in the current study, as shown in Figure 3-17c.
Figure 3-17 Scanning electron microscope image cross-section of (a) pipe specimens without corrosion, (b) pipe specimens exposed to soil of pH 2.5 for 365 days (c) pipe specimens exposed to soil of pH 3.5 for 365 days and (d) a 37-year-old grey cast iron pipe

For the sake of comparison, the microstructure photography of one exhumed old cast iron pipe (37 years old) is presented in Figure 3-17d, which shows that a resembling morphology of material deterioration was observed in the exhumed old pipe wall cross-sections. Compared with pipe exposed to corrosion for a relatively short time, a thicker layer graphitisation zone resulted in the old pipe. Again, the similarity of the material morphology of the laboratory corroded pipe and the realistic pipe corroded
in the field shows the effectiveness of reproducing the corrosion of the pipe in the laboratory using the testing methodology of this study. Overall, the results from the current study are not only useful for the research community, but also can help asset managers implement better management of pipelines through accurate corrosion assessments.

3.8 Summary

A comprehensive experimental program has been presented in this chapter to investigate the corrosion of buried cast iron pipes and its effect on their mechanical behaviour. Relatively long-term corrosion tests have been conducted on cast iron pipe sections in real soil with different conditions, as represented by different pH values of soil. Microstructure analyses of the pipe specimens before and after corrosion have also been conducted to identify the possible causes for the degradation of their mechanical properties. From the analysis of the test results, it has been found that both fracture toughness and modulus of rupture of pipe specimens decrease with corrosion over time. Further, the reduction of fracture toughness is more sensitive to corrosion-induced mass loss than that of modulus of rupture. It has also been found that localised corrosion and the formation of a graphitised zone in cast iron pipes are primary causes for the degradation of their mechanical properties. The results of pipe specimens presented in the test are in agreement with exhumed pipes. It can be concluded that the developed methodology can produce reasonable results on corrosion and its effect on degradation of mechanical properties of cast iron pipes buried in soil, which can contribute to the body of knowledge of corrosion behaviour in realistic soil and its effect on mechanical properties.
Chapter 4: Stress Intensity Factor of Corroded Pipes

4.1 Introduction

The effects of corrosion on the structural integrity of pipelines have been explored by various researchers (e.g., Ahammed, 1998; American Petroleum Institute, 1999; Kiefner & Vieth, 1989; Watkins & Anderson, 1999). An early investigation of the corrosion effect on pipe capacity was conducted by the Pipeline Research Committee of the American Gas Association (Kiefner & Vieth, 1989). This research comprised pressuring an extensive series of ex-service pipes, for which a semi-empirical equation for the residual strength of the corroded pipe was derived. Bai and Hauch (2001) investigated the bearing capacity of pipes with constant depth corrosion under combined loads and derived several analytical solutions. To capture the effects of corrosion pits on the structural capacity of pipes in a three-dimensional manner, some researchers (e.g., Ahmmad & Sumi, 2010, Cerit et al., 2009; Kolios et al., 2014) employed FE methods to investigate the stress distribution around corrosion pits. Several equations for the stress concentration factors for corroded pipes were developed (e.g., Kolios et al., 2014).

Although these formulas are useful for pipeline design and assessment in general, an inspection of in-service cast iron pipe failures reveals that the most common failures of cast iron pipelines are fracture related (Doyle et al., 2003; Makar et al., 2001) due to the brittle nature of cast iron material. It is well-known that cast iron is characterised by the presence of graphite flakes in the matrix of iron. As a result, a sharp tip often forms on the surface of corrosion pits (Conlin & Baker, 1991). Subsequently, the collapse of pipes occurs when the fracture toughness of the pipe is exceeded. Despite the practical significance of this observation, it is noted that little research into fracture mechanics has been conducted to investigate the effects of corrosion pits on the residual capacity of cast iron pipes. As reviewed in Chapter 2, although a variety of geometries have been used to represent corrosion pits, such as hemispherical pit (Cerit et al., 2009), semi-ellipsoid (Kolios et al., 2014), circular cone (Ahmmad & Sumi, 2010), a truncated cone or a parallelepiped with rounded corners (Vodka, 2015), the sharp corrosion pits could not be reasonably represented by these geometrical models.
The intention of this chapter is to investigate the effects of sharp corrosion pits on SIFs of cast iron pipes. A three-dimensional geometrical model is proposed to represent the sharp pits induced by localised corrosion. The J-integral domain method is employed in a three-dimensional FE framework to derive solutions to analytical equations for the SIFs of pipes with sharp pits. After the developed FE model is verified, the SIFs for external corrosion pits of various dimensions (i.e., widths, depths and lengths) in cast iron pipes under internal pressure are investigated. An attempt is also made to derive formulas for the maximum SIF of pipes, incorporating loading and geometrical parameters of the pipes and corrosion pits. The results presented provide important knowledge about the SIFs for cast iron pipes with the presence of corrosion pits.

The work presented in this chapter has already been published in an international journal, *Engineering Failure Analysis* (Wang et al., 2017).

### 4.2 Geometrical Model of Sharp Corrosion

Pitting corrosion is a localised form of corrosion, by which pits with a wide range of configurations are produced in the material. Kolios et al. (2014) studied the effect of corrosion pits on stress concentration factors in a plate by using a semi-ellipsoidal cavity with various aspect ratios (i.e., the ratio of pit depth to half-length: \( \frac{d}{c} \)). In a study by Nakai et al. (2006), the corrosion pit was assumed to be a circular cone (see Figure 4-1d). More recently, a truncated cone model was proposed by Setsika et al. (2015) in the simulation of a pit in microscale for the study of corroded aluminium specimens based on metallographic analysis. In some other studies, corrosion pits are modelled by simpler geometries (e.g., a rounded box or half spheres; Vodka, 2015). Typical geometrical models used in literature for the approximation of corrosion pits are summarised in Figure 4-1.
In contrast, sharp pits attract little attention compared with blunt pits. Based on fracture mechanics theory, a narrow and sharp corrosion pit with an elongated and irregular opening (as illustrated in Figure 4-2) is more critical than a blunt pit because it acts as an initial crack in cast iron pipes, leading to subsequent pipe cracking. When the root radius of the pit is sufficiently small, compared with the depth of the pit, the pit front can reasonably be assumed to be sharp and considered a re-entrant corner, defined as a corner in which the inside angle is less than 90°.
To generate such a three-dimensional cavity with a re-entrant corner, consider two identical semi-ellipsoids, which overlap at the beginning. Let the origin of the Cartesian coordinate system be the centre of the ellipsoids. The main axes of the ellipsoids are $A$, $B$ and $C$ in $x$-, $y$- and $z$-axis, respectively. When the semi-ellipsoids are shifted from the origin with a distance $b/2$ in the opposite direction along the $x$-
axis, a cavity with the re-entrant corner (sharp front) is created as shown in Figure 4-3. With various dimensions of the two semi-ellipsoids, the intersected cavity is expected to have various geometries with a specific length, width and depth. The intersected geometry is represented by a discontinuous function as follows:

\[
\begin{align*}
\left\{ \begin{array}{l}
\left( \frac{x-A-b}{2} \right)^2 + \frac{y^2}{B^2} + \frac{z^2}{C^2} = 1 & \text{for } x \geq 0 \\
\left( \frac{x-A+b}{2} \right)^2 + \frac{y^2}{B^2} + \frac{z^2}{C^2} = 1 & \text{for } x < 0
\end{array} \right.
\end{align*}
\]

(Equation 4.1a)

\[
\varphi = 2 \arccos \left( 1 - \frac{b}{2A} \right)
\]

(Equation 4.1b)

\[
A = C = \frac{a^2}{b} + \frac{b}{4}
\]

(Equation 4.1c)

\[
B = \frac{ac}{b} + \frac{bc}{4a}
\]

(Equation 4.1d)

In Equation 4, \(A, B\) and \(C\) are the main axes of the ellipsoids in \(x\)-, \(y\)- and \(z\)-axis, respectively. Further, \(a\) is the pit depth, \(b\) is the pit width, \(c\) is the half-length of the pit and \(\varphi\) is the opening angle of the re-entrant corner at the deepest point. The merit of this equation is that it can generate corrosion pits with various shapes and dimensions. This is important because in reality, corrosion pits are irregular and their size varies. A schematic of the sharp corrosion pit is displayed in Figure 4-4.
Figure 4-3 Schematic for the generation of a sharp corrosion pit

Figure 4-4 A pressurised pipe with an external sharp corrosion pit
4.3 Formulation of the Stress Intensity Factor

When sharp corrosion pits appear on the pipe surface (as shown in Figure 4-4), the stress analysis based on classical strength theory becomes invalid due to the stress singularity induced by the discontinuities. Instead, SIF, $K$, based on fracture mechanics theory needs to be employed to quantify the asymptotic stress distribution close to a re-entrant corner (Lazzarin & Tovo, 1996). For a specific structure, $K$ is not only related to the applied stresses, but also a function of the overall geometry of the structure and crack. In general, there are three deformation modes of fracture (Li & Yang, 2012): (1) opening mode (Mode I); (2) in-plane shear mode (Mode II); and (3) out-of-plane shear or tear mode (Mode III). In this work, only Mode I is considered since it is the dominant cracking condition in pipes under normal service conditions. In general, the Mode I SIF $K_I$ can be expressed as follows (Hertzberg, 1996):

$$K_I = \sigma_0 \sqrt{\pi a f(\theta)}$$  \hspace{1cm} (Equation 4.2)

In Equation 4.2, $K_I$ is the Mode I SIF, $\sigma_0$ is the applied stress, $a$ is the pit depth, $\theta$ is the angle that defines the position of points along the front of a re-entrant corner and $f(\theta)$ is the influence coefficient function allowing for the various dimensions of the structure and pit.

As known in the literature (e.g., Raju & Newman, 1982), the influence coefficient function is related to the depth of the pit ($a$), the length of the pit ($2c$) and the dimensions of the pipe. It is known (Williams, 1952) that the SIF is associated with the opening angle of the re-entrant corner ($\phi$), which is associated with the pit width ($b$), based on Equation 4.1b). As a result, the influence coefficient is a function of pit width, $b$. Accordingly, the SIF for pipes subjected to corrosion pits can be determined as follows:

$$K_I = \sigma_0 \sqrt{\pi a f\left(\frac{a}{c}, \frac{a}{d}, \frac{d}{R}, \frac{b}{c}, \theta\right)}$$  \hspace{1cm} (Equation 4.3)

In Equation 4.3, $\sigma_0$ is the applied stress, $f$ is the influence coefficient function, $d$ is the wall thickness of the pipe and $R$ is the inner radius of the pipe. When a pipe is
subjected to internal pressure \( (p) \), the average hoop stress \( (\sigma_0) \) can be represented as follows (Raju & Newman, 1982):

\[
\sigma_0 = \frac{pR}{d}
\]  

(Equation 4.4)

### 4.4 Three-Dimensional Finite Element Modelling

#### 4.4.1 Formulation of Influence Coefficient Function

It is widely accepted that it is extremely difficult to derive the SIF \( (K) \) analytically for structures with complex geometries. The numerical approaches based on the FE method have proved to have a reasonable accuracy (Atluri & Kathiresan, 1980; Li & Yang, 2012; Raju & Newman, 1982). In general, there are two approaches that can be used to determine SIFs: the displacement-based method and the energy-based J-integral method. The accuracy of displacement-based methods relies on the accuracy of the stress and displacement field; a very fine meshing around the pit front is required in the FE model. Alternatively, the numerical method based on the J-integral (Li et al., 2016b; Li & Yang, 2012) is the most accurate means for determining the SIFs for structures with cracks. In comparison, the energy-based J-integral method is theoretically path-independent and can produce accurate results with relatively coarse meshes. Therefore, the energy-based J-integral method is employed in the current study to determine SIFs.

According to Griffith’s (1920) energy theory, for a given body (e.g., a pipe), the change of its total potential energy \( (d\Pi) \), associated with a virtual crack extension \((da)\) can be expressed as follows (Mohammadi, 2008):

\[
-\frac{\partial \Pi}{\partial a} = \frac{\partial W}{\partial a} - \frac{\partial U^e}{\partial a}
\]  

(Equation 4.5)

In Equation 4.5, \( W \) is the work supplied by the applied external load, \( U^e \) is the elastic component of strain energy stored in the body and \(-\partial \Pi/\partial a \) can be determined based on the J-integral as follows (Hertzberg, 1996):

\[
-\frac{\partial \Pi}{\partial a} = J = \int_{\Gamma} \left( kdy - T \frac{\partial n}{\partial x} ds \right)
\]  

(Equation 4.6)
In Equation 4.6, \( \Gamma \) is a contour that encloses the crack tip, \( k \) is the strain energy density (i.e., loading work per unit volume of the elastic body), \( ds \) is the increment of the contour path, \( \vec{T} \) is the outward traction vector on \( ds \) and \( \vec{u} \) is the displacement vector at \( ds \). \( J \) physically represents the change of the total potential energy associated with a virtual crack extension (ASTM International, 2012a). For linear elastic materials, SIF \( K_I \) can be determined as follows (Hertzberg, 1996):

\[
K_I = \sqrt{J E}
\]

(Equation 4.7)

In Equation 4.7, \( \bar{E} = E \) for the plane stress problem and \( \bar{E} = E / (1 - \nu^2) \) for plane strain and triaxial stress problems (Dassault Systèmes, 2007). \( E \) is Young’s modulus and \( \nu \) is Poisson’s ratio.

Based on Equations 4.3–4.7, the influence coefficient function \( f \) can be determined:

\[
f\left(\frac{a}{c}, \frac{a}{d}, \frac{b}{c}, \frac{b}{d}, \theta\right) = \frac{K_I}{\sigma_0 \sqrt{\pi a}} = \frac{1}{\sigma_0} \frac{\sqrt{JE}}{\pi a}
\]

(Equation 4.8)

### 4.4.2 Three-Dimensional Model and Verification

The domain integral method, which is available in Abaqus 6.14 software (Dassault Systèmes Simulia Corporation, 2014), is employed in the current study to derive the solutions for \( K_I \). Since the domain integral is taken from a domain of elements around the front of the corrosion pit, the accuracy of integration relies less on the size of mesh compared with the contour integral method.

To evaluate the integral of Equation 4.6, a domain of elements in the form of rings around the pit front is created, as shown in Figure 4-5. The 20-node hexahedron elements, with reduced integration points, are employed for the rings. The first block of elements in the sharp front is ‘wedge’ elements, which are transformed from a 20-node quadratic hexahedron element by collapsing one side of the element and shifting the mid-side node to one-quarter of the element edge length from the sharp front. The usage of sophisticated singular elements (i.e., wedge elements) in the domain integral method is to improve the accuracy of the \( J \)-integral further. The second ring of elements share the same nodes with the elements in the first block. Subsequently, the next ring of elements share nodes with the elements in the
previous ring. These four rings together, with the first block of elements, form the domains for integration. The final results from the second to fifth domains are averaged, while the result from the first domain is ignored. Due to the complexity of the mesh around the pit region, 10-node quadratic tetrahedron elements are employed in the irregular region close to the pit front, while the other regions are meshed with 20-node quadratic hexahedron elements. The tie constraint is applied on the interfaces of mixed elements types.

![Figure 4-5 Domains of elements at the front of the sharp corrosion pit](image)

It is well-known that the key to obtaining an expression of SIF $K_1$ for cast iron pipes, as shown in Equation 4.3, is to derive the influence coefficient function $f$. Before determining the influence coefficient function $f$ by the three-dimensional FE model developed herein, it is necessary to verify its accuracy. It would be ideal that SIF data were available for pipes with corrosion pits either from field or laboratory tests. However, this appears impossible based on the data available in the literature. Therefore, the limited numerical data for pipes with surface cracks from literature (Li & Yang, 2012; Raju & Newman, 1982) are used for comparison.

In the numerical studies of Raju and Newman (1982), the pipe had a semielliptical surface crack. A model of one-eighth of the pipe segment, with a longitudinal crack on the surface of the pipe, was employed. The methodology starts with evaluating the
influence coefficients of SIFs for cracks under constant, linear, quadratic and cubic stress distributions, respectively. With the influence coefficients determined, the SIFs of pipes under internal pressure were calculated based on the Taylor series expansion of hoop stress. Li and Yang (2012) employed a similar approach; both low and high aspect ratio cases were studied.

For comparison, the same pipes as those of Raju and Newman (1982) and Li and Yang (2012) are examined in the current study. Due to symmetry in the x- and z-direction, one-quarter of a pipe segment is modelled, as shown in Figure 4-6. Nodes located at the symmetry plane are restrained in these directions. The stress-free condition is imposed at the ends of the pipe and the nodes with minimum y-coordinate value in the pipe are fixed. The meshing technique is employed and the pipe is subjected to internal pressure, which is applied directly to the inner surface of the pipe.

The SIFs for cracks with aspect ratios 1.0 and 2.0 are compared with those presented in Raju and Newman (1982) and Li and Yang (2012), respectively. The comparisons are presented in Figure 4-7, showing that the SIFs calculated by the current method
are in good agreement with those in Raju and Newman (1982) and Li and Yang (2012). It is noted that the maximum difference of SIF along the crack front is less than 4.6%, which suggests that the SIFs obtained from the $J$-integral method, in combination with FE method, are reliable and accurate.

![Graph comparing stress intensity factors along the crack front](a)

![Graph comparing stress intensity factors along the crack front](b)

**Figure 4-7** Comparison of stress intensity factors along the crack front for (a) $a/c = 1.0$, $d/R = 0.1$ and $b = 0$, and (b) $a/c = 2.0$, $d/R = 0.1$ and $b = 0$

With the three-dimensional FE model verified, a series of three-dimensional models with various geometries of pipes and sharp pits are developed to determine SIFs for pipes. The pressure is directly applied to the inner surface of the pipe. The geometry of the pit is modelled by Equation 4.1 and the pit front is longitudinally oriented.
Since the induced hoop stress is perpendicular to the centre plane of the sharp pit, the opening fracture mode (i.e., Mode I) will occur. For all pipe analyses, Poisson’s ratio is taken to be 0.23 for cast iron (Coffin, 1950). The behaviour of the pipe is assumed to be linear elastic, since cast iron is a brittle material with small strain at failure. As shown in Figure 4-4, the pipe has an inner diameter ($R$), wall thickness ($d$), pit depth ($a$), pit length ($2c$) and pit width ($b$). Figure 4-6 shows the mesh discretisation (15,480 hexahedron elements and 23,025 tetrahedron elements) and model dimensions. The length of the pipe under analysis is taken to be more than 10 times the corrosion pit length to reduce the boundary effects. The considered pipe covers a range of representative commercial cast iron pipes with $d/R = 0.05, 0.1$ and $0.2$. Both low aspect ratio ($a/c = 0.4, 0.6$ and $0.8$) and high aspect ratio ($a/c = 1.0, 1.6$ and $2.0$) are considered. Three values for the pit width to pit half-length ratio ($b/c$) are taken (i.e., $b/c = 0, 0.25$ and $0.5$). The ratio of the pit depth to the wall thickness ($a/d$) has values of $0.2, 0.5$ and $0.8$. In total, a parametric study with 162 different cases was conducted for calculating SIF. The parameters of the pipe and pit geometries are shown in Table 4-1.

Table 4-1 Variables in parametric finite element studies

<table>
<thead>
<tr>
<th>Variable</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a/c$</td>
<td>0.4</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
</tr>
<tr>
<td>$a/d$</td>
<td>0.2</td>
</tr>
<tr>
<td>$b/c$</td>
<td>0</td>
</tr>
<tr>
<td>$d/R$</td>
<td>0.05</td>
</tr>
</tbody>
</table>

4.5 Effect of Geometric Properties on Stress Intensity Factor

The influence coefficients for pressurised cast iron pipes with various geometries of sharp pits have been calculated using Abaqus software (Dassault Systèmes Simulia Corporation, 2014) and Equation 4.8. For a cylinder with a semielliptical surface crack, the main geometrical parameters that affect the SIFs are (1) relative depth ratio ($a/d$), (2) the aspect ratio ($a/c$) and (3) $d/R$. The SIF varies along the crack front. Details of the effect of these ratios on the SIF are evident in the literature (e.g., Atluri & Kathiresan, 1980; Li & Yang, 2012; Raju & Newman, 1982). In this paper, the proposed geometrical model for corrosion pits has not only a re-entrant corner, as in
a semielliptical surface crack, but also a width perpendicular to the sharp front. A thorough review of the literature suggests that this study is the first time that the effect of a sharp pit on the SIFs for pipes has been investigated. The influence coefficients were calculated at 91 nodes distributed uniformly along the pit front. The angle $\theta$ determines the position of each node. The distributions of influence coefficients (i.e., values of $f$) along the pit front as a function of the angle $\theta$ for various values of $alc$ and $b/c$ with $d/R = 0.1$ and $ald = 0.5$ are shown in Figure 4-8. The figures for the other sets of $d/R$ and $ald$ can be produced, but are omitted here due to length limit of the paper.

Figure 4-8 shows that the general trend of influence coefficient distributions considerably varies from pipes with low aspect ratio pits (i.e., $alc = 0.4$) to those with high aspect ratio ones (i.e., $alc = 2.0$). For pits with low aspect ratios, the influence coefficient increases gradually from the surface point ($\theta = 0$) to the deepest point ($\theta = 90$), as shown in Figure 4-8a. The distribution of influence coefficients in the cases of high aspect ratio has an inverse trend; the maximum value occurs near the surface point and gradually decreases from the surface point to the deepest point. This is consistent with the findings in the cylindrical vessels with semielliptical surface cracks (Atluri & Kathiresan, 1980; Li & Mahmoodian, 2013; Li & Yang, 2012; Raju & Newman, 1982). Although the influence coefficient distribution demonstrates the representative trend of low aspect ratio when $alc = 0.4$, this representative pattern of distribution fades when the aspect ratio increases to 0.8, wherein the influence coefficients at the deepest and surface points of pits are larger than those at other positions (shown in Figure 4-8b). When the aspect ratio exceeds 0.8, maximum SIFs occur at the position closest to the surface point of the pit (i.e., when $\theta$ is approximately $3^\circ$).
Figure 4-8 Influence coefficients along pit front for (a) $a/c = 0.4$, $d/R = 0.1$ and
$a/d = 0.5$; (b) $a/c = 0.8$, $d/R = 0.1$ and $a/d = 0.5$; (c) $a/c = 2.0$, $d/R = 0.1$ and
$a/d = 0.5$
It is noted that the pit width, $b$, has considerable effects on influence coefficients. Generally, influence coefficients decrease when the $b/c$ value increases from 0 to 0.5, as shown in Figure 4-8. It is observed that the geometry of the corrosion pit becomes a surface crack when $b/c$ is 0 (i.e., the pit front becomes extremely sharp), which results in a high-stress singularity around the pit front. Additionally, it is noted that the difference in the influence coefficients for different values of $b/c$ is not proportional to the difference in $b/c$. Further, it is important to note that the smaller the aspect ratio $a/c$ is, the larger effect the pit width has on the influence coefficient. For example, in Figure 4-8a, the influence coefficients along the pit front are reduced by 9.0% and 40.0% on average when the ratio of width to half-length ($b/c$) increases from 0 to 0.25 and from 0 to 0.5, respectively. While in the case of high aspect ratio, as observed in Figure 4-8c, the influence coefficient decreases by 2.1% and 3.9% on average when the $b/c$ increases from the same 0 to 0.25 and 0 to 0.5, respectively. This finding implies that pits with low aspect ratio are more sensitive to $b/c$.

The effects of $b/c$ on the reduction of maximum influence coefficients are further investigated and compared with that of other geometrical parameters, as shown in Figure 4-9. A comprehensive summary of the maximum influence coefficients for each given parameter of $d/R$, $a/d$, $b/c$ and $a/c$ are presented in Table 4-2 to Table 4-4. In Figure 4-9a, it is noted that the maximum influence coefficient decreases rapidly as the $b/c$ increases in the case of $a/c = 0.4$. However, for pipes with high aspect ratio pits (as shown in Figure 4-9b) with $a/c = 2.0$, the reduction of the maximum influence coefficient is relatively small.
Figure 4-9 Effect of pit width on maximum influence coefficient for (a) \( \frac{a}{c} = 0.4 \)
and \( \frac{d}{R} = 0.2 \) and (b) \( \frac{a}{c} = 1.6 \) and \( \frac{d}{R} = 0.2 \)
Table 4-2 Maximum influence coefficients for pipes with $d/R = 0.05$

<table>
<thead>
<tr>
<th>$a/c$</th>
<th>$a/d$</th>
<th>$b/c$</th>
<th>$\theta$ (degree)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.2</td>
<td>0.5</td>
<td>0.8</td>
</tr>
<tr>
<td>0.4</td>
<td>0.943</td>
<td>1.136</td>
<td>1.259</td>
</tr>
<tr>
<td>0.4</td>
<td>0.868</td>
<td>0.808</td>
<td>1.168</td>
</tr>
<tr>
<td>0.4</td>
<td>0.558</td>
<td>0.676</td>
<td>0.873</td>
</tr>
<tr>
<td>0.6</td>
<td>0.844</td>
<td>0.923</td>
<td>1.105</td>
</tr>
<tr>
<td>0.6</td>
<td>0.811</td>
<td>0.890</td>
<td>1.041</td>
</tr>
<tr>
<td>0.6</td>
<td>0.672</td>
<td>0.725</td>
<td>0.931</td>
</tr>
<tr>
<td>0.8</td>
<td>0.761</td>
<td>0.851</td>
<td>1.013</td>
</tr>
<tr>
<td>0.8</td>
<td>0.732</td>
<td>0.810</td>
<td>0.964</td>
</tr>
<tr>
<td>0.8</td>
<td>0.678</td>
<td>0.742</td>
<td>0.878</td>
</tr>
<tr>
<td>1.0</td>
<td>0.735</td>
<td>0.814</td>
<td>0.928</td>
</tr>
<tr>
<td>1.0</td>
<td>0.702</td>
<td>0.774</td>
<td>0.891</td>
</tr>
<tr>
<td>1.0</td>
<td>0.702</td>
<td>0.717</td>
<td>0.819</td>
</tr>
<tr>
<td>1.6</td>
<td>0.676</td>
<td>0.721</td>
<td>0.774</td>
</tr>
<tr>
<td>1.6</td>
<td>0.657</td>
<td>0.691</td>
<td>0.743</td>
</tr>
<tr>
<td>1.6</td>
<td>0.635</td>
<td>0.654</td>
<td>0.699</td>
</tr>
<tr>
<td>2.0</td>
<td>0.639</td>
<td>0.665</td>
<td>0.701</td>
</tr>
<tr>
<td>2.0</td>
<td>0.620</td>
<td>0.644</td>
<td>0.677</td>
</tr>
<tr>
<td>2.0</td>
<td>0.603</td>
<td>0.616</td>
<td>0.644</td>
</tr>
</tbody>
</table>
Table 4-3 Maximum influence coefficients for pipes with \( d/R = 0.1 \)

<table>
<thead>
<tr>
<th>( a/c )</th>
<th>( a/d )</th>
<th>( b/c )</th>
<th>( \theta ) (degree)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.4</td>
<td>0.2</td>
<td>0.926</td>
<td>1.142</td>
</tr>
<tr>
<td>0.4</td>
<td>0.5</td>
<td>0.836</td>
<td>1.007</td>
</tr>
<tr>
<td>0.4</td>
<td>0.8</td>
<td>0.545</td>
<td>0.680</td>
</tr>
<tr>
<td>0.6</td>
<td>0.2</td>
<td>0.810</td>
<td>0.917</td>
</tr>
<tr>
<td>0.6</td>
<td>0.5</td>
<td>0.810</td>
<td>0.896</td>
</tr>
<tr>
<td>0.6</td>
<td>0.8</td>
<td>0.661</td>
<td>0.730</td>
</tr>
<tr>
<td>0.8</td>
<td>0.2</td>
<td>0.747</td>
<td>0.845</td>
</tr>
<tr>
<td>0.8</td>
<td>0.5</td>
<td>0.727</td>
<td>0.804</td>
</tr>
<tr>
<td>0.8</td>
<td>0.8</td>
<td>0.666</td>
<td>0.735</td>
</tr>
<tr>
<td>1.0</td>
<td>0.2</td>
<td>0.728</td>
<td>0.822</td>
</tr>
<tr>
<td>1.0</td>
<td>0.5</td>
<td>0.696</td>
<td>0.771</td>
</tr>
<tr>
<td>1.0</td>
<td>0.8</td>
<td>0.650</td>
<td>0.722</td>
</tr>
<tr>
<td>1.6</td>
<td>0.2</td>
<td>0.683</td>
<td>0.719</td>
</tr>
<tr>
<td>1.6</td>
<td>0.5</td>
<td>0.654</td>
<td>0.691</td>
</tr>
<tr>
<td>1.6</td>
<td>0.8</td>
<td>0.624</td>
<td>0.653</td>
</tr>
<tr>
<td>2.0</td>
<td>0.2</td>
<td>0.622</td>
<td>0.664</td>
</tr>
<tr>
<td>2.0</td>
<td>0.5</td>
<td>0.637</td>
<td>0.642</td>
</tr>
<tr>
<td>2.0</td>
<td>0.8</td>
<td>0.594</td>
<td>0.614</td>
</tr>
</tbody>
</table>
An analysis of the data in Tables 4-2 to Table 4-4 shows that the ratio of pipe thickness to pipe radius \((d/R)\) has a slight effect on the maximum influence coefficient, with a maximum difference 3.46%. It suggests that the stress gradients around sharp pit fronts have a weak dependence on pipe dimensions, compared with the parameters of the pit itself. As expected, it is observed that the ratio of pit depth to pipe thickness \((a/d)\) has a great effect on the maximum influence coefficients, which confirms that SIFs of pipes become large when the corrosion depths increase.

From these results, it has been shown that the pit width \((b)\) considerably affects SIFs, especially for pipes with low aspect ratios. The results suggest that assuming pitting corrosion is a surface crack will result in inaccurate estimates of SIFs. In addition, the maximum SIF occurs at the deepest point along the pit front for pits with low aspect ratios (e.g., \(a/c = 0.4\)), while it occurs near the surface point of the pit when the aspect ratio of the pit is high (e.g., \(a/c = 2.0\)). This suggests that the position of

<table>
<thead>
<tr>
<th>(a/c)</th>
<th>(a/d)</th>
<th>(b/c)</th>
<th>(\theta) (degree)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.4</td>
<td>0.2</td>
<td>0.905</td>
<td>1.141 1.458 0</td>
</tr>
<tr>
<td>0.4</td>
<td>0.5</td>
<td>0.758</td>
<td>1.331 0.25 0</td>
</tr>
<tr>
<td>0.4</td>
<td>0.8</td>
<td>0.525</td>
<td>0.886 0.5 0</td>
</tr>
<tr>
<td>0.6</td>
<td>0.2</td>
<td>0.811</td>
<td>0.921 1.136 0</td>
</tr>
<tr>
<td>0.6</td>
<td>0.5</td>
<td>0.780</td>
<td>0.887 1.070 0.25</td>
</tr>
<tr>
<td>0.6</td>
<td>0.8</td>
<td>0.646</td>
<td>0.726 0.908 0.5</td>
</tr>
<tr>
<td>0.8</td>
<td>0.2</td>
<td>0.722</td>
<td>0.820 1.005 0</td>
</tr>
<tr>
<td>0.8</td>
<td>0.5</td>
<td>0.701</td>
<td>0.780 0.952 0.25</td>
</tr>
<tr>
<td>0.8</td>
<td>0.8</td>
<td>0.650</td>
<td>0.713 0.863 0.5</td>
</tr>
<tr>
<td>1.0</td>
<td>0.2</td>
<td>0.692</td>
<td>0.802 0.943 0</td>
</tr>
<tr>
<td>1.0</td>
<td>0.5</td>
<td>0.667</td>
<td>0.755 0.898 0.25</td>
</tr>
<tr>
<td>1.0</td>
<td>0.8</td>
<td>0.630</td>
<td>0.698 0.824 0.5</td>
</tr>
<tr>
<td>1.6</td>
<td>0.2</td>
<td>0.654</td>
<td>0.707 0.769 0</td>
</tr>
<tr>
<td>1.6</td>
<td>0.5</td>
<td>0.624</td>
<td>0.6755 0.730 0.25</td>
</tr>
<tr>
<td>1.6</td>
<td>0.8</td>
<td>0.595</td>
<td>0.634 0.695 0.5</td>
</tr>
<tr>
<td>2.0</td>
<td>0.2</td>
<td>0.599</td>
<td>0.644 0.696 0</td>
</tr>
<tr>
<td>2.0</td>
<td>0.5</td>
<td>0.597</td>
<td>0.629 0.672 0.25</td>
</tr>
<tr>
<td>2.0</td>
<td>0.8</td>
<td>0.577</td>
<td>0.597 0.635 0.5</td>
</tr>
</tbody>
</table>
the fracture propagation could be different for pipes with corrosion pits of different aspect ratios. As a result, the fracture failure modes will be different. These results have not been found in previous literature.

4.6 Modelling of Maximum Stress Intensity Factor

To predict the remaining service life of cast iron pipes with sharp pits, a predictive model of maximum SIFs as a function of pipe and pit geometries is required. A recently developed data regression approach, named evolutionary polynomial regression (EPR), has been proven to be efficient in constructing the complicated relationship between a set of independent variables and the output results (Giustolisi & Savic, 2009). The EPR constructs symbolic models by genetic algorithm as the first stage and estimates constant values by the least squares method as the second stage (Giustolisi & Savic, 2006). The main advantage of the EPR is that it not only returns regression models with reasonably accurate prediction of results, but also allows for a scientific understanding of some underlying mechanisms. One of the typical pseudo-polynomial expressions in the EPR allows the construction of the following form:

\[ Y = a_0 + \sum_{j=1}^{m} a_j \cdot X_1 \cdot \ldots \cdot X_k g(X_1) \cdot \ldots \cdot g(X_k) \quad \text{(Equation 4.9)} \]

In Equation 4.9, \( Y \) is the estimated output matrix of the system, \( a_0 \) is an optional bias term, \( a_j \) is the constant value to be estimated, \( X_1 \ldots X_k \) are input vector of variables, \( k \) is the number of independent predictor variables (input) and \( g \) is the inner function defined by the user, based on the preknowledge of the problem. The key idea of the EPR is to start from Equation 4.9 and search for the best form of the function (i.e., a combination of vectors of independent variables) and then perform the least squares regression to find the adjustable parameters \( a_j \) for each combination of inputs.

In the current study, the EPR is employed to derive the relationship between the influence coefficient function and geometrical properties of the structural components. Both non-negative least squares numerical method and multi-objective optimisation strategy for the evolutionary search of EPR are applied: (1) the minimisation of the number of terms and (2) the minimisation of the number of inputs and the maximisation of model accuracy. Through trial and error, the
exponential function is found the most accurate inner function in the construction of Equation 4.9. The influence coefficients for maximum SIFs, as shown in Tables 4-2 to 4-4, are used as the predictor variables. The results of the EPR method predictions and target values (i.e., predictor variables) are shown in Figure 4-10a for high aspect ratio cases and Figure 4-10b for low aspect ratio cases. Since the maximum SIFs occur at different locations for pipes with high aspect ratio pits and low aspect ratio pits, two formulas are constructed for each respectively as follows:

\[
f\left(\frac{a}{c}, \frac{a}{d}, \frac{d}{R}, \frac{b}{c}\right) = \xi_1 \frac{c}{\sqrt{ad}} e^{\frac{d}{2R}} + \xi_2 \sqrt{\frac{d}{a}} e^{\frac{b}{2R}} + \xi_3 \frac{b}{2R}
\]

(Equation 4.10a)

\[
f\left(\frac{a}{c}, \frac{a}{d}, \frac{d}{R}, \frac{b}{c}\right) = \xi_1 \sqrt{\frac{c}{a}} e^{\frac{d}{2R}} + \xi_2 \sqrt{\frac{d}{a}} e^{\frac{b}{2R}} + \xi_3 \frac{b}{2R}
\]

(Equation 4.10b)

The range of parameters over which Equation 4.10a is valid is \(1.0 \leq \frac{al}{c} \leq 2.0\), \(0.2 \leq \frac{ald}{c} \leq 0.8\), \(0 \leq \frac{bl}{c} \leq 0.5\), \(0.05 \leq \frac{d}{R} \leq 0.2\). The range of parameters over which Equation 4.10b is valid is \(0.4 \leq \frac{al}{c} \leq 0.8\), \(0.2 \leq \frac{ald}{c} \leq 0.8\), \(0 \leq \frac{bl}{c} \leq 0.5\), \(0.05 \leq \frac{d}{R} \leq 0.2\). Both formulas have concise forms and appear to effectively explain the role of the geometrical properties of the pit and pipe on influence coefficients based on this study’s observations. The formulas of influence coefficient functions have three terms and three constants (i.e., \(\xi_1\), \(\xi_2\) and \(\xi_3\)). Estimates for these are shown in Table 4-5. All input variables are expressed as pipe and pit geometrical parameters.

<table>
<thead>
<tr>
<th>Table 4-5 Constants in Equation 4.10 and the fitting indicators</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\xi_1)</td>
</tr>
<tr>
<td>(f(\frac{al}{c} \geq 1.0))</td>
</tr>
<tr>
<td>(f(\frac{al}{c} \leq 0.8))</td>
</tr>
</tbody>
</table>

Note: CoD is coefficient of determination; AVG is average error; and SSE is sum of squared residuals.
Figure 4-10 Comparison of results predicted by the evolutionary polynomial regression method and target values for (a) high aspect ratio case and (b) low aspect ratio case

The fit of the equations was examined by the coefficient of determination (CoD). The CoD indicates the proportion of the variance that is predictable from the independent variable (Chang, Wang & Du, 2009). That is, $\text{CoD} = 1 - \frac{\text{SSE}}{\text{SST}}$ with $\text{CoD} = 1$ being a perfect model fit and wherein the sum of squares of residuals (SSE) represents the unexplained part of variance, in relation to the total sum of squares (SST). SSE is used to guide the search for the best fit model. Details of this search have been presented by Giustolisi and Savic (2006). As SST is proportional to the variance of data, the average error (AVG) is also used as a measure of how well the
equation replicates the target values. The CoD, SSE and AVG of Equation 4.10 are presented in Table 4-5. It is noted that Equation 4.10 agrees well with the whole set of data, with a CoD at 98.1% and 94.0% for high aspect ratio and low aspect ratio, respectively. This suggests that 98.1% of the variability has been accounted for by Equation 4.10a and 94.0% by Equation 4.10b.

According to the definition of SIFs (as shown in Equation 4.3), the maximum SIF $K_{IM}$ for $a/c \geq 1.0$ and $a/c \leq 0.8$ are respectively expressed as follows:

$$K_{IM} = \sigma_0 \sqrt{\pi a} \left( \frac{c}{\xi_1} \frac{d}{\sqrt{ad}} e^{\frac{d}{2R}} \frac{a - b}{2c} + \frac{d}{\sqrt{a}} e^{\frac{d}{2R}} \frac{e^{-\frac{b}{2c}} + \xi_3 e^{\frac{b}{2c}}}{c} \right)$$

(Equation 4.11a)

$$K_{IM} = \sigma_0 \sqrt{\pi a} \left( \frac{c}{\xi_1} \frac{d}{\sqrt{ad}} e^{\frac{d}{2R}} \frac{a - b}{2c} + \frac{d}{\sqrt{a}} e^{\frac{d}{2R}} \frac{e^{-\frac{b}{2c}} + \xi_3 e^{\frac{b}{2c}}}{c} \right)$$

(Equation 4.11b)

The final expressions of maximum SIFs are functions of the applied stress, pipe and pit geometrical parameters. Compared with other published literature (Li & Mahmoodian, 2013; Yang et al., 2013), the formulas developed herein incorporate the influence of pit width on the SIF of the pipe. Additionally, the proposed expressions for the maximum SIFs have more concise forms, even compared with those developed for cylinders with surface crack (Raju & Newman, 1982).

To verify the developed formula (Equation 4.11), the SIFs determined by Equation 4.11 and those from literature and FE models have been compared. Since there is no data for three-dimensional pits in the reported literature (e.g., Atluri & Kathiresan, 1980; Li & Yang, 2012; Raju & Newman, 1982), the Equation 4.11 formulas are used for comparison in surface crack cases by taking $b = 0$. In this circumstance, the sharp corrosion pit is equivalent to a semielliptical surface crack. The geometries of the pit and pipe that are taken from the literature are $d = 10$ mm, $R = 100$ mm, $a/c = 0.4, 1.0$ and $2.0$, and $a/d = 0.2, 0.5$ and $0.8$. The internal pressure ($p$) is assumed to be 1 Mpa. The comparison of the maximum SIFs determined both by Equation
4.11 and the data from the literature (Li & Yang, 2012; Raju & Newman, 1982) is shown in Figure 4-11.

![Figure 4-11 Comparison of maximum stress intensity factors, calculated from the derived formulas and literature](image)

Figure 4-11 Comparison of maximum stress intensity factors, calculated from the derived formulas and literature

It is noted that the maximum SIFs determined by the developed formulas are generally in good agreement with those from the literature, with an average difference of 4.32%. As expected, maximum SIFs increase when the pit depth increases. For a given pit depth, the maximum SIFs of pipe with low aspect ratio pits (i.e., $a/c = 0.4$) is larger than those with high aspect ratio pits (i.e., $a/c = 1.0$ and $a/c = 2.0$). Equation 4.11 has also been verified against FE generated data for $b/c > 0$. In the FE models, the geometries of the pit are $a/d = 0.35$ and 0.65, $a/c = 0.6$ and 1.6, and $b/c = 0.25$ and 0.5. The results of the comparison are shown in Figure 4-12. It can be observed that the proposed equation can reasonably predict the maximum SIFs. Clearly, the equation proposed in the current study is useful for engineers to predict the service life of aged pipes with sharp corrosion pits.
Figure 4.12 Comparison of maximum stress intensity factors calculated from finite element (FE) models and the derived formulas
4.7 Summary

SIFs for pipes with sharp corrosion pits have been evaluated in this chapter. A new three-dimensional geometrical model for corrosion pits has been proposed. The domain integral method has been employed, in conjunction with a three-dimensional FE analysis, to derive the SIFs for pipes with sharp pits. After verification of the FE model, SIFs for pipes with sharp corrosion pits of various geometrical parameters have been investigated. An expression for the maximum SIF has been developed for corrosion pits in cast iron pipes by EPR. It has been found that the distributions of SIFs for pits with low and high aspect ratios are different and the maximum SIF occurs at the surface point of corrosion pits when the aspect ratio $a/c$ exceeds 0.8. It has also been found that the influence of pit width on the SIF is substantial and that the influence of pit width on SIFs for low aspect ratio pits is larger than that for high aspect ratio pits. It can be concluded that assuming corrosion pits are surface cracks will result in an overestimation of the SIF, causing an inaccurate prediction of the risk of pipe failure due to pitting corrosion. The current research provides a useful tool for engineers and asset managers to accurately assess the failure of cast iron pipelines affected by corrosion.
Chapter 5: Statistical Analysis on Field Corrosion Data

5.1 Introduction

Due to the existence of mineral salts and water content, soil is generally assumed to be a good electrolyte for corrosion (Pritchard et al., 2013). Recently, the literature has indicated that soil is responsible for 65% of buried iron corrosion (stray currents and contact with other metals account for 35%; Kreysa & Schütze, 2008; Romer & Bell, 2001). The most common type of damage is external corrosion (Alamilla et al., 2009). Although the corrosion in soil essentially follows the electrochemical theory, the analyses of pipe corrosion in soils are often complicated because of the various chemical agents and the complexity of the heterogeneity of soils. Hence, it is difficult to correlate the basic electrochemical theories with the actual corrosion of buried iron in most circumstances (e.g., Petersen et al., 2013; Moore & Hallmark, 1987; Norin & Vinka, 2003; Romanoff, 1957, 1964). Moreover, most of the soil parameters (e.g., moisture, temperature and resistivity) change spatially and temporally. Thus, soil corrosivity is normally difficult to categorise due to surrounding soil properties and pipe materials (Ferreira et al., 2007).

Based on the literature review in Chapter 2, it has been identified that corrosion was a function of many factors (e.g., soil moisture, resistivity, oxygen level, soil permeability, clay content and pH), not a singular soil parameter (Moore & Hallmark, 1987). Although the reported research work is helpful to understand the corrosion of buried metal pipes, in most cases, very weak correlations between soil properties and corrosion depth growth were observed (e.g., Doyle et al., 2003; Norin & Vinka, 2003) and contradictory conclusions regarding the effect of soil properties on corrosion (e.g., soil resistivity) still exist among research community (Gupta & Gupta, 1979; Murray & Moran, 1989; Norin & Vinka, 2003). Consequently, the understanding of the cause and effect relationship in the corrosion of buried iron is still uncertain and the prediction of corrosion growth is often subjective.

One of the main reasons for the lack of effective correlation between corrosion and soil properties is insufficient information reported on corrosion evolution and the relevant soil properties (e.g., Doyle et al., 2003; Petersen & Melchers, 2012; Moore
& Hallmark, 1987). It is known that electrochemical activities occurring at the metal surface are greatly affected by processes such as rainfall, the diffusion of chemical agents, water and oxygen transport and processes within oxides (Cole & Marney, 2012). With the absence of a comprehensive collection of data—such as local meteorological information, soil texture and chemical content—across a large sample size, it is extremely difficult to develop conclusive knowledge about how and to which degree these factors affect the corrosion behaviour of buried pipes. Further, in many field survey tests (e.g., Alamilla et al., 2009; Caleyo et al., 2009; Petersen & Melchers, 2012), the breakdown time of pipe protection coating is unavailable, which can bring additional uncertainties to analyses. Thus, a considerable number of samples with detailed information on backfill condition and metal corrosion are required to interpret the dependence of corrosion growth on soil properties.

Another possible reason for the research gap in the correlation between corrosion growth and soil properties is that soils were rarely categorised or analysed in the past, until statistical methods were employed to investigate the dependence of corrosion growth on soil properties. It is well-known that soil type can greatly determine not only the soil–liquid and soil–air interactions (Cole & Marney, 2012), but that the presence of chemical agents (such as salt and organic content) is also involved (Ferreira et al., 2007). This may explain the lack of strong correlations between soil properties and corrosion growth in the published literature, wherein the analyses were conducted by investigating a soil sample as a whole, not a combination of parts.

The intention of this chapter is to investigate the dependence of cast iron pipe corrosion on soil properties. A historical corrosion database is thoroughly examined based on published information about the growth of corrosion pits and the accompanying detailed soil properties. The correlations among soil properties are statistically analysed and the corrosion data are classified into four groups: good, fair, poor and very poor aeration. The significant influence of aeration on corrosion growth pattern is verified and the dependence of corrosion on soil properties is thoroughly investigated for each group. The results produced from this study can enrich current knowledge about the corrosion of metal pipes buried in soils and thus, facilitate engineers and asset managers to mitigate the risk of failures of metal pipes.
The work presented in this chapter has been accepted for publication in an international journal, *Journal of Materials in Civil Engineering* (Wang et al., in press).

### 5.2 Methodology for Analysis

The methodology proposed in this chapter is schematically shown in Figure 5-1. The methodology comprises data mining, corrosion data regression analysis, soil classification and verification and correlation analysis.

![Flowchart of the proposed methodology](image)

*Figure 5-1 Flowchart of the proposed methodology*
5.2.1 Data Mining

A detailed review of literature shows that there are a limited number of databases available regarding the corrosion of buried cast iron pipes, apart from few reported studies (e.g., Alamilla et al., 2009; Doyle et al., 2003; Petersen et al., 2013; Romanoff, 1957). A comparison of these databases shows that the NBS database (Romanoff, 1957) is particularly of use, as it contains successive corrosion measurements for grey cast iron pipes, which experienced a relatively long-term corrosion (i.e., 9–17 years), and a detailed recording of the accompanying soil information. More significantly, the grey cast iron pipe specimens in the NBS database have the same or similar material composition, allowing a good comparison of corrosion behaviour in different soil environments. Moreover, the test locations selected were free of stray currents and unlike other field research (e.g., Alamilla et al., 2009; Doyle et al., 2003; Petersen et al., 2013), the pipe samples buried in the NBS program were uncoated, which can eliminate the uncertainty associated with the damage of coating.

A field burial corrosion test program was conducted by the US NBS between 1922 and 1952 to investigate the performance of the commonly used metals and alloys in various locations throughout the US. Pipe sections, with diameters from 38 mm to 152 mm, were buried at 128 different locations for a corrosion exposure period of up to 19 years. Most of corrosion measurement data and soil properties information were documented by Romanoff (1957); however, in this chapter, they will be referred to as the NBS database. Although some of the NBS data were correlated by empirical (deterministic) relationships by Romanoff (1957), no attempt was made to statistically investigate the effects of soil properties on corrosion and most of the data were left for engineers to judge (Rossum, 1969).

In this study, the corrosion measurements of grey cast iron pipes and the accompanying detailed soil information from 73 different locations, documented by the NBS database, are used for analysis. According to Romanoff (1957), two to three batches of pipe sections (150 mm in diameter and 300 mm in length) were buried in each location, with most of the pipes subjected to 17 years of corrosion exposure. In each batch, two duplicates were excavated approximately every two years and the maximum corrosion depths were measured for each pipe section. A total of 2,036...
corrosion measurements on grey cast iron pipe sections are collated for analysis in the current study. To represent the unique corrosion characteristics of each soil sample, the corrosion measurements from different batches are averaged. For example, Table 5-1 shows a sample of the (averaged) successive corrosion measurements of cast iron pipes in soils with different aeration levels. The accompanying soil property information, together with meteorological data (e.g., annual precipitation and temperature), are collected, as illustrated in Table 5-2. The soils used in this study include 18 soils with good aeration, 19 soils with fair aeration, 23 soils with poor aeration and 7 soils with very poor aeration. Aeration is used as a broad term to describe the extent of voids in soil (Romanoff, 1957, 1964; Rossum, 1969). The classification of soil aeration in the NBS database was through internal drainage tests and the criterion of classification was based on the suction of soil, according to the Soil Survey Manual (United States Bureau of Plant Industry, Soils and Agricultural Engineering, 1951) as shown in Table 5-3.

<table>
<thead>
<tr>
<th>Soil Number</th>
<th>Duration of Corrosion (Year)</th>
<th>Max Corrosion on Depth (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.0</td>
<td>0.97</td>
<td></td>
</tr>
<tr>
<td>3.6</td>
<td>2.31</td>
<td></td>
</tr>
<tr>
<td>5.5</td>
<td>2.46</td>
<td></td>
</tr>
<tr>
<td>7.7</td>
<td>4.18</td>
<td></td>
</tr>
<tr>
<td>9.6</td>
<td>4.73</td>
<td></td>
</tr>
<tr>
<td>11.6</td>
<td>4.9</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.1</td>
<td>1.04</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>1.07</td>
<td></td>
</tr>
<tr>
<td>5.9</td>
<td>1.80</td>
<td></td>
</tr>
<tr>
<td>7.9</td>
<td>2.37</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>1.97</td>
<td></td>
</tr>
<tr>
<td>17.6</td>
<td>2.97</td>
<td></td>
</tr>
<tr>
<td>Soil Number</td>
<td>Duration of Corrosion (Year)</td>
<td>Max Corrosion Depth (mm)</td>
</tr>
<tr>
<td>-------------</td>
<td>----------------------------</td>
<td>--------------------------</td>
</tr>
<tr>
<td></td>
<td>1.9</td>
<td>0.56</td>
</tr>
<tr>
<td></td>
<td>4.1</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td>6.2</td>
<td>1.96</td>
</tr>
<tr>
<td></td>
<td>8.1</td>
<td>1.88</td>
</tr>
<tr>
<td></td>
<td>12.1</td>
<td>1.98</td>
</tr>
<tr>
<td></td>
<td>17.5</td>
<td>3.47</td>
</tr>
<tr>
<td>5</td>
<td>1.1</td>
<td>1.68</td>
</tr>
<tr>
<td></td>
<td>3.8</td>
<td>1.74</td>
</tr>
<tr>
<td></td>
<td>5.8</td>
<td>2.18</td>
</tr>
<tr>
<td></td>
<td>7.7</td>
<td>2.07</td>
</tr>
<tr>
<td></td>
<td>9.9</td>
<td>4.90</td>
</tr>
<tr>
<td></td>
<td>11.8</td>
<td>6.08</td>
</tr>
<tr>
<td>8</td>
<td>1</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>3.5</td>
<td>0.95</td>
</tr>
<tr>
<td></td>
<td>5.5</td>
<td>0.74</td>
</tr>
<tr>
<td></td>
<td>7.7</td>
<td>1.03</td>
</tr>
<tr>
<td></td>
<td>11.5</td>
<td>1.85</td>
</tr>
<tr>
<td></td>
<td>16.9</td>
<td>2.82</td>
</tr>
<tr>
<td>9</td>
<td>1.9</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>4.1</td>
<td>1.17</td>
</tr>
<tr>
<td></td>
<td>6.2</td>
<td>1.16</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>0.51</td>
</tr>
<tr>
<td></td>
<td>12.1</td>
<td>1.88</td>
</tr>
<tr>
<td></td>
<td>17.5</td>
<td>2.08</td>
</tr>
<tr>
<td>12</td>
<td>1.9</td>
<td>0.81</td>
</tr>
<tr>
<td></td>
<td>4.2</td>
<td>1.47</td>
</tr>
<tr>
<td></td>
<td>5.9</td>
<td>0.81</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>3.38</td>
</tr>
<tr>
<td>13</td>
<td>1.2</td>
<td>0.73</td>
</tr>
<tr>
<td></td>
<td>3.8</td>
<td>1.94</td>
</tr>
<tr>
<td></td>
<td>5.8</td>
<td>2.63</td>
</tr>
<tr>
<td></td>
<td>7.7</td>
<td>0.51</td>
</tr>
<tr>
<td></td>
<td>9.8</td>
<td>2.95</td>
</tr>
<tr>
<td></td>
<td>11.7</td>
<td>2.84</td>
</tr>
<tr>
<td>Soil Number</td>
<td>Duration of Corrosion (Year)</td>
<td>Max Corrosion on Depth (mm)</td>
</tr>
<tr>
<td>-------------</td>
<td>------------------------------</td>
<td>-----------------------------</td>
</tr>
<tr>
<td>23</td>
<td>1.9</td>
<td>2.95</td>
</tr>
<tr>
<td></td>
<td>4.3</td>
<td>6.59</td>
</tr>
<tr>
<td></td>
<td>6.2</td>
<td>8.04</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>7.05</td>
</tr>
<tr>
<td></td>
<td>10.2</td>
<td>7.84</td>
</tr>
<tr>
<td></td>
<td>12.1</td>
<td>9.68</td>
</tr>
<tr>
<td>25</td>
<td>1</td>
<td>0.64</td>
</tr>
<tr>
<td></td>
<td>3.7</td>
<td>0.51</td>
</tr>
<tr>
<td></td>
<td>5.7</td>
<td>2.04</td>
</tr>
<tr>
<td></td>
<td>7.6</td>
<td>2.54</td>
</tr>
<tr>
<td></td>
<td>11.7</td>
<td>2.77</td>
</tr>
<tr>
<td></td>
<td>17</td>
<td>3.10</td>
</tr>
<tr>
<td>26</td>
<td>1</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>3.5</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td>5.5</td>
<td>1.21</td>
</tr>
<tr>
<td></td>
<td>7.7</td>
<td>1.63</td>
</tr>
<tr>
<td></td>
<td>11.5</td>
<td>3.32</td>
</tr>
<tr>
<td></td>
<td>16.9</td>
<td>2.70</td>
</tr>
<tr>
<td>32</td>
<td>1</td>
<td>0.64</td>
</tr>
<tr>
<td></td>
<td>3.7</td>
<td>1.06</td>
</tr>
<tr>
<td></td>
<td>5.8</td>
<td>1.17</td>
</tr>
<tr>
<td></td>
<td>7.6</td>
<td>3.33</td>
</tr>
<tr>
<td></td>
<td>9.6</td>
<td>1.84</td>
</tr>
<tr>
<td></td>
<td>11.7</td>
<td>2.73</td>
</tr>
<tr>
<td>33</td>
<td>1</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>3.7</td>
<td>1.11</td>
</tr>
<tr>
<td></td>
<td>5.8</td>
<td>1.27</td>
</tr>
<tr>
<td></td>
<td>7.6</td>
<td>3.78</td>
</tr>
<tr>
<td></td>
<td>9.7</td>
<td>3.82</td>
</tr>
<tr>
<td></td>
<td>11.7</td>
<td>4.35</td>
</tr>
<tr>
<td>35</td>
<td>1.9</td>
<td>0.74</td>
</tr>
<tr>
<td></td>
<td>4.1</td>
<td>0.84</td>
</tr>
<tr>
<td></td>
<td>6.2</td>
<td>0.64</td>
</tr>
<tr>
<td></td>
<td>10.2</td>
<td>0.95</td>
</tr>
<tr>
<td></td>
<td>12.1</td>
<td>1.09</td>
</tr>
<tr>
<td></td>
<td>17.5</td>
<td>2.11</td>
</tr>
<tr>
<td>Soil Number</td>
<td>Duration of Corrosion (Year)</td>
<td>Max Corrosion on Depth (mm)</td>
</tr>
<tr>
<td>-------------</td>
<td>-----------------------------</td>
<td>----------------------------</td>
</tr>
<tr>
<td>44</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.1</td>
<td>0.42</td>
</tr>
<tr>
<td></td>
<td>3.6</td>
<td>1.57</td>
</tr>
<tr>
<td></td>
<td>5.7</td>
<td>1.08</td>
</tr>
<tr>
<td></td>
<td>7.6</td>
<td>1.13</td>
</tr>
<tr>
<td></td>
<td>11.6</td>
<td>1.74</td>
</tr>
<tr>
<td>53</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.56</td>
</tr>
<tr>
<td></td>
<td>5.4</td>
<td>1.71</td>
</tr>
<tr>
<td></td>
<td>7.5</td>
<td>2.72</td>
</tr>
<tr>
<td></td>
<td>9.4</td>
<td>3.01</td>
</tr>
<tr>
<td></td>
<td>14.4</td>
<td>3.60</td>
</tr>
<tr>
<td>59</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>5.1</td>
<td>0.53</td>
</tr>
<tr>
<td></td>
<td>7.2</td>
<td>0.53</td>
</tr>
<tr>
<td></td>
<td>9.1</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td>14.2</td>
<td>0.87</td>
</tr>
<tr>
<td>60</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td>5.6</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td>7.7</td>
<td>1.46</td>
</tr>
<tr>
<td></td>
<td>9.6</td>
<td>1.77</td>
</tr>
<tr>
<td></td>
<td>14.4</td>
<td>1.82</td>
</tr>
<tr>
<td>61</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.81</td>
</tr>
<tr>
<td></td>
<td>4.1</td>
<td>1.40</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>2.65</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>2.16</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>2.12</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>2.25</td>
</tr>
<tr>
<td>67</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>2.76</td>
</tr>
<tr>
<td></td>
<td>5.3</td>
<td>3.48</td>
</tr>
<tr>
<td></td>
<td>7.3</td>
<td>3.81</td>
</tr>
<tr>
<td></td>
<td>9.2</td>
<td>3.95</td>
</tr>
<tr>
<td></td>
<td>14.3</td>
<td>3.95</td>
</tr>
</tbody>
</table>

Source: Romanoff (1957).
Table 5-2 Sample of soil property information from National Bureau of Standards database

<table>
<thead>
<tr>
<th>Soil No.</th>
<th>Resistivity Ω cm, 15.5 °C</th>
<th>Type</th>
<th>Aeration</th>
<th>Location</th>
<th>Mean Temperature (°C)</th>
<th>pH</th>
<th>Composition of Water Extract, mg per 100 g of Soil</th>
<th>Moisture Equivalent (%)</th>
<th>Air-Pore Space (%)</th>
<th>Annual Precipitation (mm)</th>
<th>Apparent Specific Gravity (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1215</td>
<td>Silt loam</td>
<td>P</td>
<td>Cleveland, OH</td>
<td>9.56</td>
<td>7</td>
<td>11.4</td>
<td>0.72</td>
<td>0.25</td>
<td>0.43</td>
<td>0.09</td>
</tr>
<tr>
<td>2</td>
<td>684</td>
<td>Clay</td>
<td>P</td>
<td>Dallas, TX</td>
<td>18.61</td>
<td>7.3</td>
<td>3.5</td>
<td>0.28</td>
<td>1.09</td>
<td>0.13</td>
<td>1.18</td>
</tr>
<tr>
<td>5</td>
<td>1345</td>
<td>Clay adobe</td>
<td>P</td>
<td>Oakland, CA</td>
<td>56.4</td>
<td>7</td>
<td>6.5</td>
<td>0.93</td>
<td>0.48</td>
<td>0.1</td>
<td>0.69</td>
</tr>
<tr>
<td>8</td>
<td>350</td>
<td>Clay loam</td>
<td>P</td>
<td>Fargo, ND</td>
<td>13.56</td>
<td>7.6</td>
<td>1.42</td>
<td>1.72</td>
<td>2.55</td>
<td>0.71</td>
<td>0.01</td>
</tr>
<tr>
<td>9</td>
<td>2820</td>
<td>Silt loam</td>
<td>P</td>
<td>Sidney, OH</td>
<td>10.67</td>
<td>6.8</td>
<td>7.2</td>
<td>3190</td>
<td>Sandy loam</td>
<td>F</td>
<td>Los Angeles, CA</td>
</tr>
<tr>
<td>12</td>
<td>3190</td>
<td>Sandy loam</td>
<td>F</td>
<td>Bakersfield, CA</td>
<td>16.89</td>
<td>9.5</td>
<td>6.23</td>
<td>0.09</td>
<td>0.13</td>
<td>1.12</td>
<td>1.64</td>
</tr>
<tr>
<td>18</td>
<td>1410</td>
<td>Silt loam</td>
<td>G</td>
<td>Omaha, NE</td>
<td>10.33</td>
<td>7.3</td>
<td>1.4</td>
<td>0.27</td>
<td>0.63</td>
<td>0.2</td>
<td>0.94</td>
</tr>
<tr>
<td>23</td>
<td>278</td>
<td>Silt loam</td>
<td>F</td>
<td>Buttonwillow, CA</td>
<td>18.33</td>
<td>9.4</td>
<td>8.38</td>
<td>0.38</td>
<td>0.22</td>
<td>1.87</td>
<td>1.12</td>
</tr>
<tr>
<td>25</td>
<td>1780</td>
<td>Clay loam</td>
<td>F</td>
<td>Milwaukee, WI</td>
<td>7.83</td>
<td>7.2</td>
<td>4.7</td>
<td>0.23</td>
<td>0.7</td>
<td>0.41</td>
<td>1.01</td>
</tr>
<tr>
<td>26</td>
<td>2980</td>
<td>Silt loam</td>
<td>G</td>
<td>Springfield, OH</td>
<td>11.67</td>
<td>7.3</td>
<td>2.6</td>
<td>0.27</td>
<td>0.5</td>
<td>0.31</td>
<td>0.7</td>
</tr>
<tr>
<td>32</td>
<td>5700</td>
<td>Loam</td>
<td>G</td>
<td>Rochester, NY</td>
<td>8.67</td>
<td>7.3</td>
<td>0.5</td>
<td>0.23</td>
<td>0.7</td>
<td>0.12</td>
<td>0.73</td>
</tr>
<tr>
<td>33</td>
<td>800</td>
<td>Pent</td>
<td>VP</td>
<td>Milwaukee, WI</td>
<td>7.83</td>
<td>6.8</td>
<td>36</td>
<td>1.52</td>
<td>7.3</td>
<td>4.06</td>
<td>2.27</td>
</tr>
<tr>
<td>35</td>
<td>2060</td>
<td>Loam</td>
<td>G</td>
<td>Los Angeles, CA</td>
<td>16.89</td>
<td>7.3</td>
<td>5.7</td>
<td>0.68</td>
<td>0.68</td>
<td>0.49</td>
<td>1.1</td>
</tr>
<tr>
<td>44</td>
<td>1000</td>
<td>Silt loam</td>
<td>G</td>
<td>Omaha, NE</td>
<td>10.33</td>
<td>5.8</td>
<td>8.8</td>
<td>1.05</td>
<td>1.08</td>
<td>0.66</td>
<td>1.97</td>
</tr>
<tr>
<td>53</td>
<td>406</td>
<td>Clay loam</td>
<td>VP</td>
<td>Atlanta, CA</td>
<td>16.22</td>
<td>7.1</td>
<td>5.1</td>
<td>3.12</td>
<td>0.69</td>
<td>0.47</td>
<td>0.8</td>
</tr>
<tr>
<td>59</td>
<td>1660</td>
<td>Muck</td>
<td>VP</td>
<td>Kalamazoo, MI</td>
<td>9.44</td>
<td>5.6</td>
<td>12.6</td>
<td>1.03</td>
<td>3.08</td>
<td>2.7</td>
<td>0</td>
</tr>
<tr>
<td>60</td>
<td>84</td>
<td>Peat</td>
<td>VP</td>
<td>Plymouth, OH</td>
<td>9.44</td>
<td>2.6</td>
<td>297.4</td>
<td>2.91</td>
<td>10.95</td>
<td>2.86</td>
<td>0</td>
</tr>
<tr>
<td>61</td>
<td>943</td>
<td>Clay</td>
<td>F</td>
<td>New Orleans, LA</td>
<td>20.72</td>
<td>6.8</td>
<td>4.9</td>
<td>0.73</td>
<td>0.68</td>
<td>0.33</td>
<td>0.71</td>
</tr>
<tr>
<td>67</td>
<td>455</td>
<td>Cinders</td>
<td>VP</td>
<td>Milwaukee, WI</td>
<td>7.83</td>
<td>7.6</td>
<td>0.77</td>
<td>3.03</td>
<td>0.53</td>
<td>0.55</td>
<td>0.08</td>
</tr>
</tbody>
</table>

Source: Romanoff (1957). Note: G denotes good, F denotes fair, P denotes poor and VP denotes very poor.
Table 5-3 Classification of soil aeration based on internal drainage tests from

Soil Survey Manual

<table>
<thead>
<tr>
<th>Groups</th>
<th>Suction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Good aeration</td>
<td>&gt; 1,500 kPa</td>
</tr>
<tr>
<td>Fair aeration</td>
<td>60–1,500 kPa</td>
</tr>
<tr>
<td>Poor aeration</td>
<td>10–60 kPa</td>
</tr>
<tr>
<td>Very poor aeration</td>
<td>&lt; 10 kPa</td>
</tr>
</tbody>
</table>


The collated soil data includes soil physical properties (e.g., air-pore space, apparent specific gravity and moisture equivalent) and chemical properties (e.g., pH, resistivity and salt compositions). Generally, all the soil physical properties are associated with soil permeability and moisture retaining capacity. Specifically, moisture equivalent is defined as the percentage of water (by weight) that a soil can retain under a centrifugal force 1,000 times that of gravity (Briggs & McLane, 1910; Romanoff, 1957). The air-pore space is defined as the percentage of the volume of soil at a definite moisture content that is occupied by air (Romanoff, 1957). The apparent specific gravity is an index of the compactness of mineral soils (Pritchard et al., 2013). The volume shrinkage is often used to quantify the tendency of the soil to crack on drying and to swell when wetted (Pritchard et al., 2013).

5.2.2 Fitting of Corrosion Data

Pitting corrosion is complicated in nature and has been intensively investigated in the past decades (Bhandari et al., 2015). Despite a limited understanding of pitting corrosion behaviour (Bhandari et al., 2015; Melchers, 2008), it is known that pitting corrosion growth rate is a nonlinear and time-dependent process. The most widely used model for the approximation of pit growth is based on a power law model, which relates the depth of pit to the exposure time as follows (Marcus, 2011):

\[ a = kt^n \]  

(Equation 5.1)

In Equation 5.1, \( a \) is the depth of pit at time (\( t \)) and \( k \) and \( n \) are pitting proportionality and exponent factors, respectively. For a given corrosion measurements over exposure time, the \( k \) and \( n \) factors can be determined by data fitting.
To cross compare the datasets from different locations and soils with various time intervals, the growth of pit depth is characterised by a pair of time-independent parameters (i.e., $k$ and $n$ factors in Equation 5.1) for each soil, as schematically shown in Figure 5-1. It is known that the area for testing is one of the most important factors affecting the correlation of the results of field exposure or laboratory tests with pipeline performance. This is because the possibility of finding a deeper pit increases when the area under observation increases. Since this study aims to investigate the influence of soil properties on corrosion rates of pipes (with same dimensions/surface area) rather than to develop a corrosion model, the area factor is not considered in Equation 5.1.

5.2.3 Classification of Soil Samples

As can be observed from literature (Logan et al., 1937; Kreysa & Schütze, 2008), the corrosion rate of ferrous metals varies from an almost unchanged high rate in some soils, to nearly ceasing in other soils. This shows that corrosion has an exclusive dependence on soil type. One of the reasons for this phenomenon is that the underlying corrosion mechanisms of metals buried in different soils are different (Norin & Vinka, 2003). According to Rossum (1969), in different aerated soils, the corrosion processes vary with diffusion of oxygen, accumulation of ferrous ions at the anode and formation of hydroxyl ions at the cathode surface. These variations directly affect the electrodes polarisation, cell potential drop and the resistance of the circuit, which finally determines the corrosion rate.

To verify whether aeration has a significant effect on corrosion growth pattern, the analysis of variance (ANOVA) based on an F-test has been conducted on corrosion model parameters, pitting proportionality ($k$) and exponent factors ($n$), in four predefined groups. The fundamental idea of ANOVA is to assess whether the expected values of $k$ or $n$ differ from each other among different aerated soil groups by comparing the F-test value with a threshold/critical value, which is determined by the sample size and the significance level defined ($\alpha = 0.05$). The F-test works by decomposing the variability of data. The ratio of mean variances between groups and within a group is used as a test statistic to accept or reject the null hypothesis that soils with various aerations produce the same corrosion growth pattern. Mathematically, F-test statistic is defined as follows (Devore, 2012):
In Equation 5.2, $K$ is the number of groups, $\bar{Y}_i$ denotes the sample mean, $\bar{Y}$ denotes the overall mean value of the sample, $Y_{ij}$ denotes the $j^{th}$ observed result in the $i^{th}$ out of $K$ groups and $N$ is the total size of sample. In Equation 5.2, the numerator donates the mean variance between groups and the denominator donates the mean variance within groups.

**5.2.4 Correlating Analysis**

The soil-based factors affecting corrosion include not only those associated with soil heterogeneity, but also chemical agents. In this chapter, the Pearson’s correlation coefficient ($r$) was employed to characterise the strength of the correlation between soil corrosion model parameters and soil properties (Doyle et al., 2003). As such, $r$ is defined as the covariance of two variables, divided by the product of their standard deviation; $r$ value ranges from $-1$ to $1$, with an absolute value closing to $1$ (indicating a strong relationship), while a value of zero means an unrelated relationship. Generally, a correlation coefficient $r < 0.5$ is regarded as a weak correlation (Rumsey, 2015).

Apart from studying the influence of soil properties on corrosion growth, the correlations between these factors were investigated. Soil properties, in most circumstances, interact with each other. To identify the direct—rather than secondary—connection of corrosion growth to a soil property, it is also necessary to investigate the correlation and dependence among soil properties. There are many chemical elements existing in soils (Romanoff, 1957), but most of them are combined in soluble compounds, which have little impact on the corrosion of iron. Thus, the current study is limited to chemicals that are soluble in water, namely sodium ($Na^+$), potassium ($K^+$), calcium ($Ca^{2+}$), magnesium ($Mg^{2+}$), carbonate ($HCO_3^-$), chloride ($Cl^-$), nitrate ($NO_3^-$) and sulphate ($SO_4^{2-}$). A correlation coefficient was again employed as an indicator of the strength of interaction among the soil properties. Specifically, the relationship between resistivity and the total weight of salts, pH value and acidity content were investigated because these factors are often recognised
as key corrosion-influencing factors. In addition to the chemical properties of soil, correlation analyses were conducted for physical properties that affect the access of oxygen to soil and the soil’s water retaining capacity. These soil properties include air-pore space, apparent specific gravity, moisture equivalent, volume shrinkage and annual precipitation. Since the soil physical factors and chemical agents are different in nature, it is reasonable to assume in this study that there is no dependence between them.

5.3 Results and Analysis

5.3.1 Maximum Pit Depth

In this chapter, the results of maximum pit depth versus time were used to fit Equation 5.1 for each soil sample. The coefficients of determination ($r^2$) obtained from all soils are presented in Figure 5-2. The analysis of the results revealed that the fitting of corrosion growth data in 60 soil locations out of 73 locations has a CoD, $r^2 > 0.8$. It is also noted that the coefficients of determination in six locations were less than 0.6, due to the scatter of measured corrosion depth at a given time. One of the reasons for this could be due to the variance induced by the spatially and temporally changing weather and soil properties in these locations. It may also be attributed to the presence of manufacturing defects in pipe material and accuracy in measuring the maximum pit depth. Overall, the analysis results imply, despite random occurrence and growth of corrosion in nature, the corrosion growth process can be described by Equation 5.1 with reasonable confidence.
5.3.2 Verification of Soil Classification

Soils were classified into different groups namely ‘good’, ‘fair’, ‘poor’ and ‘very poor’ aeration and ‘all’. The mean and coefficient of variance (COV) of \( k \) and \( n \) values in different soil groups were calculated, as shown in Table 5-4. To verify the selection of the soil’s aeration classification, the ANOVA was conducted on both \( k \) and \( n \), as summarised in Table 5-5. It can be noted from Table 5-4 that the mean value of both \( k \) and \( n \) are different between different groups. In terms of \( n \), Table 5-5 demonstrates that the F-value (5.161) is larger than the critical F-value (2.737), implying that soil aeration level causes the mean exponent factor (\( n \)) to be different between groups. In comparison, the F-value for \( k \) (0.673) is smaller than the critical F-value (2.737), which reveals that aeration is not the determining factor for the variance of \( k \) value.

<table>
<thead>
<tr>
<th>Groups</th>
<th>( k ) Mean</th>
<th>( k ) COV</th>
<th>( n ) Mean</th>
<th>( n ) COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Good aeration</td>
<td>0.858</td>
<td>0.850</td>
<td>0.765</td>
<td>0.292</td>
</tr>
<tr>
<td>Fair aeration</td>
<td>0.820</td>
<td>0.660</td>
<td>0.522</td>
<td>0.532</td>
</tr>
<tr>
<td>Poor aeration</td>
<td>0.860</td>
<td>0.835</td>
<td>0.718</td>
<td>0.379</td>
</tr>
<tr>
<td>Very poor aeration</td>
<td>0.373</td>
<td>0.379</td>
<td>0.945</td>
<td>0.177</td>
</tr>
</tbody>
</table>
Table 5-5 Analysis of variance (ANOVA) results of $n$ and $k$ in different soil groups

<table>
<thead>
<tr>
<th>Source of Variation</th>
<th>SS</th>
<th>df</th>
<th>MS</th>
<th>$F$</th>
<th>$P$-value</th>
<th>F crit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Between groups</td>
<td>1.018</td>
<td>3</td>
<td>0.339</td>
<td>5.161</td>
<td>0.003</td>
<td>2.737</td>
</tr>
<tr>
<td>Within groups</td>
<td>4.536</td>
<td>69</td>
<td>0.066</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>5.554</td>
<td>72</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Source of Variation</th>
<th>SS</th>
<th>df</th>
<th>MS</th>
<th>$F$</th>
<th>$P$-value</th>
<th>F crit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Between groups</td>
<td>0.870</td>
<td>3</td>
<td>0.290</td>
<td>0.673</td>
<td>0.572</td>
<td>2.737</td>
</tr>
<tr>
<td>Within groups</td>
<td>29.758</td>
<td>69</td>
<td>0.431</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>30.628</td>
<td>72</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: SS denotes the total sum of squares, df denotes degree of freedom and MS denotes mean/scaled sum of squares.

Based on fundamental electrochemical theory, Rossum (1969) proposed the hypothesis that the aeration of soil greatly affects the formation of corrosion pits in ferrous metals, which determines the initial corrosion rate and growth pattern. Unfortunately, the work of Rossum’s has not been validated; this is primarily due to the scarcity of corrosion data with detailed recordings of soil information. By performing ANOVA analysis among four groups of soils with different aeration levels, the effect of aeration on corrosion has been statistically verified, which validates Rossum’s work statistically and hence, adds new knowledge to this field.

5.3.3 Effects of Soil Properties on $k$ and $n$ Factors

After verifying the classification of soil samples, the effects of soil properties on proportionality ($k$) and exponent ($n$) factors were investigated in five groups of soils (i.e., all, good, fair, poor and very poor). The results are shown in Table 5-6. The soil properties investigated include resistivity, pH, total acidity, moisture equivalent, air-pore space, apparent specific gravity and volume shrinkage. Since the in situ water content of soil in the field was not available, the annual precipitation data in each location was used to take the water content into account. These soil properties are considered to represent the averaged exposure conditions considering that they fluctuate over time within a small interval of values in nature (Romanoff, 1957).
<table>
<thead>
<tr>
<th>Property</th>
<th>All</th>
<th>Good Aeration</th>
<th>Fair Aeration</th>
<th>Poor Aeration</th>
<th>Very Poor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$k$</td>
<td>$n$</td>
<td>$k$</td>
<td>$n$</td>
<td>$k$</td>
</tr>
<tr>
<td>Resistivity</td>
<td>0.070</td>
<td>-0.313</td>
<td>0.365</td>
<td>-0.286</td>
<td>-0.267</td>
</tr>
<tr>
<td>Log of resistivity</td>
<td>-0.274</td>
<td>-0.178</td>
<td>0.367</td>
<td>-0.235</td>
<td>-0.598</td>
</tr>
<tr>
<td>pH</td>
<td>0.301</td>
<td>0.009</td>
<td>-0.209</td>
<td>-0.073</td>
<td>0.440</td>
</tr>
<tr>
<td>Total acidity</td>
<td>0.129</td>
<td>0.111</td>
<td>0.299</td>
<td>-0.074</td>
<td>0.575</td>
</tr>
<tr>
<td>Annual precipitation</td>
<td>-0.379</td>
<td>-0.034</td>
<td>0.430</td>
<td>-0.073</td>
<td>-0.523</td>
</tr>
<tr>
<td>Moisture equivalent</td>
<td>0.126</td>
<td>0.065</td>
<td>0.546</td>
<td>-0.002</td>
<td>0.175</td>
</tr>
<tr>
<td>Air-pore space</td>
<td>-0.131</td>
<td>-0.112</td>
<td>-0.183</td>
<td>-0.444</td>
<td>-0.117</td>
</tr>
<tr>
<td>Apparent specific gravity</td>
<td>-0.094</td>
<td>0.248</td>
<td>-0.148</td>
<td>0.257</td>
<td>-0.065</td>
</tr>
<tr>
<td>Volume shrinkage</td>
<td>0.048</td>
<td>0.062</td>
<td>-0.033</td>
<td>0.443</td>
<td>-0.230</td>
</tr>
</tbody>
</table>

Note: Correlation coefficient of $r < 0.5$ is generally regarded as a weak correlation (Rumsey, 2015).
It is noted from Table 5-6 that generally, the effects of soil parameters on \( k \) and \( n \) are different in various groups of soils. Specifically, in soils with good aeration, it can be observed that the moisture equivalent has the highest correlation coefficient \((r = 0.546)\) with \( k \) factor; resistivity and annual precipitation have relatively weak correlation coefficients \((r = 0.365 \text{ and } 0.430, \text{ respectively})\). Conversely, all soil properties have very weak correlations with \( n \) factor, except air-pore space \((r = -0.444)\) and volume shrinkage \((r = 0.443)\).

In soils with fair aeration, three soil parameters—log of resistivity, total acidity and annual precipitation—have been found to have relatively high correlation coefficients with \( k \) \((r = 0.598, 0.575 \text{ and } -0.523, \text{ respectively})\). However, no strong correlation has been found between soil properties and \( n \), with exception of a weak correlation between \( n \) and total acidity \((r = -0.2)\).

In soils with poor aeration, several soil properties—log of resistivity, annual precipitation and volume shrinkage—were identified to have relatively high correlation coefficients with \( k \) \((r = 0.607, -0.630 \text{ and } 0.555, \text{ respectively})\). In terms of factor \( n \), like the observations in the ‘good aeration’ and ‘fair aeration’ groups, the \( n \) factor has a weak correlation with all soil properties.

In soils with very poor aeration, \( k \) has high correlation coefficients with pH and annual precipitation \((r = -0.984 \text{ and } 0.523, \text{ respectively})\). A relatively weak correlation has been observed between \( k \) and total acidity \((r = -0.297)\), and \( k \) and moisture equivalent \((r = -0.305)\). With regards to \( n \) factor, it is observed that \( n \) has high correlation coefficients with annual precipitation \((r = -0.847)\) and moisture equivalent \((r = 0.876)\), while relatively weak correlations with other soil properties, such as log of resistivity \((r = 0.362)\), pH \((r = 0.399)\) and total acidity \((r = 0.431)\).

From the results, it can also be observed that the log of soil resistivity has a higher correlation coefficient with \( k \) and \( n \) than soil resistivity in different soil groups. This result is in reasonable agreement with the results of Doyle et al. (2003). This implies that some soil properties (e.g., soil resistivity) are nonlinearly correlated with corrosion rate. Incorporating the nonlinear characteristics of soil properties into statistical correlation analysis can increase the accuracy of identifying key corrosion-influencing factors.
5.3.4 Correlation between Soil Properties

It is known that dissolved salts are the primary factors that determine soil resistivity (Doyle et al., 2003; Moore & Hallmark, 1987; Nürnberger, 2012). The dissolved salts that were documented in the NBS database include Na\(^+\), K\(^+\), Ca\(^{2+}\), Mg\(^{2+}\), HCO\(_3\)\(^-\), Cl\(^-\) and SO\(_4\)\(^{2-}\). The correlation between resistivity and the content of salts was statistically investigated, as shown in Table 5-7. Analysis results reveal that there are relatively weak correlations between soil resistivity and each type of chemical agent. It has been found that Na\(^+\) and K\(^+\) have the highest correlation coefficient \((r = -0.347)\) with soil resistivity, while there is no correlation between HCO\(_3\)\(^-\) and resistivity \((r = -0.081)\). The correlation coefficients among different chemicals indicate that salt agents exist in soils most likely in forms of sodium chloride (NaCl), potassium chloride (KCl), sodium sulphate (Na\(_2\)SO\(_4\)), potassium sulphate (K\(_2\)SO\(_4\)), calcium sulphate (CaSO\(_4\)), magnesium sulphate (MgSO\(_4\)) and magnesium chloride (MgCl\(_2\)). For example, Na\(^+\) and K\(^+\) has a high correlation coefficient with Cl\(^-\) \((r = 0.821)\); the correlation coefficient between Ca\(^{2+}\) and SO\(_4\)\(^{2-}\) is as high as 0.803.

<table>
<thead>
<tr>
<th></th>
<th>Resistivity</th>
<th>Total Weight</th>
<th>Na(^+) &amp; K(^+)</th>
<th>Ca(^{2+})</th>
<th>Mg(^{2+})</th>
<th>HCO(_3)(^-)</th>
<th>Cl(^-)</th>
<th>SO(_4)(^{2-})</th>
</tr>
</thead>
<tbody>
<tr>
<td>Resistivity</td>
<td>1.000</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total weight</td>
<td>-0.372</td>
<td>1.000</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Na(^+) &amp; K(^+)</td>
<td>-0.347</td>
<td>0.922</td>
<td>1.000</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ca(^{2+})</td>
<td>-0.310</td>
<td>0.770</td>
<td>0.541</td>
<td>1.000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mg(^{2+})</td>
<td>-0.275</td>
<td>0.693</td>
<td>0.529</td>
<td>0.538</td>
<td>1.000</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HCO(_3)(^-)</td>
<td>-0.081</td>
<td>-0.237</td>
<td>-0.147</td>
<td>-0.181</td>
<td>-0.344</td>
<td>1.000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cl(^-)</td>
<td>-0.239</td>
<td>0.736</td>
<td>0.821</td>
<td>0.311</td>
<td>0.504</td>
<td>-0.222</td>
<td>1.000</td>
<td></td>
</tr>
<tr>
<td>SO(_4)(^{2-})</td>
<td>-0.330</td>
<td>0.890</td>
<td>0.697</td>
<td>0.803</td>
<td>0.648</td>
<td>-0.257</td>
<td>0.385</td>
<td>1.000</td>
</tr>
</tbody>
</table>

The correlations between moisture equivalent, air-pore space, apparent specific gravity and volume shrinkage have been analysed, as presented in Table 5-8. Moisture equivalent has a weak correlation with air-pore space \((r = -0.296)\) and a high correlation coefficient with volume shrinkage \((r = 0.747)\). It is also observed that air-pore space has a relatively strong correlation with apparent specific gravity, with a correlation coefficient of \(r = -0.641\). Overall, the general strong correlations obtained
between these soil properties are not unexpected, as all of them are associated with the permeability and moisture retention ability.

Table 5-8 Correlation between soil physical properties

<table>
<thead>
<tr>
<th></th>
<th>Moisture Equivalent</th>
<th>Air-Pore Space</th>
<th>Apparent Specific Gravity</th>
<th>Volume Shrinkage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture equivalent</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Air-pore space</td>
<td>−0.296</td>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Apparent specific gravity</td>
<td>0.185</td>
<td>−0.641</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Volume shrinkage</td>
<td>0.747</td>
<td>−0.433</td>
<td>0.458</td>
<td>1</td>
</tr>
</tbody>
</table>

5.4 Observation and Discussion

From the results, further observation and discussion can be made. For the sake of comparison, the correlation between soil properties and corrosion depth without soil classification is presented in Table 5-6 (i.e., all soil groups as identified as one group called ‘all’). From the results shown in Table 5-6, it can be noted that the correlation coefficients \( r \) between corrosion depth and soil properties in the all group are lower, compared with other groups. For example, the correlation coefficient between total acidity and \( k \) factor was found 0.129 in the all soil group. However, in fair and poor aeration soils, the correlation coefficients can be as high as 0.575 and 0.464, respectively. It needs to be noted that the magnitude of correlation produced in the all soil cluster is close to most published data (e.g., Cole & Marney, 2012; Doyle et al., 2003). That is, the correlation between corrosion and soil properties is often weak. It can be observed from the results of this study that a classification of soil, based on aeration, can produce relatively significant correlations between soil properties and corrosion growth pattern. This can help identify key factors that influence corrosion in buried cast iron pipes in different types of soils.

To illustrate the statistical features of \( k \) and \( n \) datasets (e.g., centre, spread, the presence of outliers and extent of any departure from the centre), the boxplot is employed as shown in Figure 5-3. The boxplot is a standardised method that displays the distribution of data through the summary of five numbers: smallest, lower fourth, median, upper fourth and largest (Devore, 2012). The lines stretching from the box
represent the variability outside the lower and upper fourth. The distance between the lower fourth and upper fourth gives a measure of spread.

![Boxplot for k and n in different groups of soils](image)

**Figure 5-3 Boxplot for (a) k in different groups of soils and (b) n in different groups of soils**

It can be observed from Figure 5-3 that both $k$ and $n$ factors have a relatively large degree of dispersion; a downward skewness is observed for all groups. Specifically, although the $k$ factor covers a wide range (i.e., from lowest 0.2 to highest 2.7 in mm), most $k$ values are less than 1.5. Compared with $k$ factor, the $n$ factor has relatively smaller dispersion, ranging from 0.5 to 0.85.
Generally, the $n$ factor increases with the decrease of aeration (i.e., from good to very poor aeration). This can be explained by the fact that soils with low aeration normally have a high level of water content, which can increase corrosion by promoting the transport of ions and the electric conductivity. However, there is no tendency observed between the $k$ factor and aeration, implying that there are additional factors—rather than aeration—affecting the $k$ factor. It is also noted that some extreme values are observed for both $k$ and $n$ factor results. This phenomenon is not unexpected considering that other factors, such as microorganisms (e.g., SRB) in soils, can cause rapid corrosion growth. Although MIC is expected to exist in natural soils, it is difficult to investigate in this chapter due to the lack of relevant information in the NBS database. It is noted that the MIC phenomenon and the behaviour of metals in soils was not understood when the NBS database was produced (i.e., from 1922). Nevertheless, the soils with very poor aeration can provide an anaerobic condition, which is suitable for the activities of microorganisms (e.g., SRB) that can greatly affect corrosion.

Additional analyses were undertaken to explore the nonlinear relationship between soil properties. For example, although the results of correlation analysis (as shown in Table 5-7) demonstrate that there is a relatively weak linear correlation ($r = -0.372$) between resistivity and salt content, it has been determined that the relationship between soil resistivity and total salt weight (mg per 100 g soil) follows a nonlinear power law (see Figure 5-4). This is not unexpected, since the resistivity of soil in the NBS database was measured on fully saturated soil at the same temperature (i.e., 15.5 °C) and the resistivity of soil therefore, mainly relies on the content of soluble salts content (i.e., total salt weight). These findings show the effect of the nonlinear relationship on the Pearson’s correlation coefficient ($r$).
Figure 5-4 Relationship between soil resistivity and total salt content

In addition, the relationships between moisture equivalent and air-pore space, volume shrinkage and moisture equivalent, and air-pore space and apparent specific gravity have been explored, as shown in Figure 5-5a, 5-5b and 5-5c, respectively. It can be observed from Figure 5-5a that the results of moisture equivalent versus air-pore space are scattered. However, a general trend of air-pore space versus moisture equivalent is clear, showing that an increase of moisture equivalent is associated with a decrease of air-pore space. This is because soils with fine texture, such as clay and silt, have many small pores and hence, easily retain more water than coarser textures (e.g., sand). Similarly, as volume shrinkage is an indication of the colloidal nature of the clay and loam particles in a soil (Romanoff, 1957), the increase of volume shrinkage (i.e., more clay constituent) will cause the increase of moisture and thus, retain the capacity of the soil, which can be quantified by moisture equivalent. It can be observed from Figure 5-5b that volume shrinkage and moisture equivalent are closely related and a nonlinear relationship exists between these two parameters. This observation is consistent with the fact that moisture equivalent has been widely used as a reliable measure of the texture of soil (Veihmeyer & Hendrickson, 1931). The plotting of soil air-pore space versus apparent specific gravity shows that a decrease in apparent specific gravity is associated with an increase of air-pore space, as shown in Figure 5-5c. It is known that air-pore space affects compactness and bulk density of
soil, which is consistent with the high correlation coefficient \( r = -0.641 \) that is observed between the soil air-pore space and apparent specific gravity (see Table 5-8).
5.5 Summary

The effects of soil properties on the corrosion behaviour of grey cast iron pipes have been investigated in this chapter. Statistical analyses have been conducted using historical data with a comprehensive record of both corrosion growth and detailed soil information. The corrosion rate of the maximum pit depth of cast iron in soil has been characterised by the proportionality ($k$) and exponent ($n$) factors of a power law, with different values of $k$ and $n$ pertaining to corrosion in various environments. The corrosion data has been classified into different groups (i.e., good, fair, poor and very poor) based on the aeration level of soil; the classification has been verified by analysis variance. It has been found that both the chemical and physical properties in realistic soils are closely correlated and most of them have nonlinear relationships with each other. It has also been found that the effects of soil parameters on $k$ and $n$ are different in different groups of soils. Stronger correlations between soil properties and corrosion are obtained when corrosion data is classified based on soil aeration,
rather than the soil as a whole. It can be concluded that a good relationship between corrosion and soil properties can be established when corrosion data is classified based on the soil aeration, which can create a new understanding about the underlying corrosion mechanisms in different soils.
Chapter 6: Reliability Assessment of Corroded Pipes

6.1 Introduction

The review of previous research in Chapter 2 shows that although the effects of corrosion on structural integrity of pipelines have been intensively investigated (e.g., Ahammed, 1998; Benjamin et al., 2007; Rajani, 2000; Seica & Packer, 2004; Yamamoto et al., 1983), most of them were based on strength theory. However, an inspection of in-service cast iron pipe failures reveals that the most common failures of cast iron pipelines are fracture related, due to the brittle nature of cast iron material (Conlin & Baker, 1991; Doyle et al., 2003; Makar et al., 2001).

Additionally, the corrosion of buried pipes is a complex process and the rates of corrosion widely vary depending on pipe materials, soils properties and environmental factors (Cole & Marney, 2012; Doyle et al., 2003; Romanoff, 1957). In most of the previous literature, the corrosion growth behaviour is assumed either to be linear, bilinear or nonlinear, represented by a deterministic power law (e.g., Ji et al., 2017; Li & Mahmoodian, 2013; Rajani & Kleiner, 2001). These simplified models often have limited applicability when characterising the evolution of corrosion growth corresponding to the spatial and temporal variability of soil properties. A corrosion model developed specifically for cast iron pipes buried in soils is still lacking.

In addition, most of the reported reliability analyses on corroded pipes were based on time-independent methods (e.g., the Monte Carlo simulation, FORM or first-order reliability method; see Ahammed, 1997; Ahammed, 1998; Ahammed & Melchers, 1996; Caley et al., 2002; Sadiq et al., 2004). A relatively limited number of studies were reported to use the application of time-dependent methods (e.g., upcrossing method), except for few (e.g., Li & Mahmoodian, 2013; Yang et al., 2018). It is known that there are many parameters related to failures of pipes, including pipe dimensions, loading conditions, corrosion geometry and pipe materials. Most of them are time-variant variables and demonstrate various extents of variations. To consider these parameters with uncertainties and time-variant characteristics, it is significant to employ a stochastic method to quantify the probability of pipe failures.
In this chapter, a methodology is developed to evaluate the probability of fracture failure in pressurised cast iron pipes, subject to pitting corrosion. The SIF of pipe is modelled as a nonstationary lognormal process, based on fracture mechanics and stochastic process theory. A probabilistic model of pit depth in buried cast iron pipes is developed, based on field data mining. A time-dependent method, based on the first passage probability theory, is employed to quantify the probability of pipe fracture failure due to pitting corrosion. A case study is presented to demonstrate the developed methodology and the sensitivity analysis is conducted to identify the key parameters that have a great effect on the probability of pipe failure.

The work presented in this chapter has been submitted for publication in an international journal.

6.2 Modelling of Stress Intensity Factor

Pitting corrosion is a localised form of corrosion. A detailed examination of the literature suggests that the research has focused more on corrosion-induced blunt pits, than on sharp pits. For example, in the works of Benjamin et al. (2000), Choi et al. (2003) and Freire et al. (2006), corrosion pits were assumed to have a rectangular shape with a smooth surface. Other geometries, such as semi-ellipsoidal cavity (Ji et al., 2015; Kolios et al., 2014) and circular cone (Ahmmad & Sumi, 2010; Nakai et al., 2006) have also been used. However, a narrow and sharp corrosion pit is recognised more critical than blunt pits because it can act as an initial crack in pipes, facilitating subsequent fracture. To simulate such a sharp corrosion pit, a cavity with the re-entrant corner (sharp front) as developed in Wang et al. (2017) is used in this study. For a pipe subjected to a sharp corrosion pit and internal pressure, the formula of SIF can be expressed as follows (Wang et al., 2017):

\[ K_1 = \frac{pR}{d} \sqrt{\pi a f \left( \frac{a}{c}, \frac{a}{d}, \frac{d}{R}, \frac{b}{c} \right)} \]  

(Equation 6.1)

In Equation 6.1, \( p \) is the internal pressure, \( a \) is the pit depth, \( f \) is the influencing coefficient function, \( c \) is the half-length of the pit, \( d \) is the wall thickness of the pipe, \( b \) is the width of the pit and \( R \) is the inner radius of the pipe. Only a Mode I fracture is
considered, since it is found to be the dominant cracking condition in pipes under normal service conditions. The influence coefficient functions in Equation 6.1 for pressurised cast iron pipes with sharp corrosion pits were determined by Wang et al. (2017) using three-dimensional FE fracture analyses:

\[ f\left(\frac{a}{c}, \frac{a}{d}, \frac{d}{R}, \frac{b}{c}\right) = \xi_1 \frac{c}{\sqrt{ad}} e^{\frac{d}{2R} - \frac{a}{2d} - \frac{b}{2c} + 1} + \xi_2 \frac{d}{a} e^{\frac{b}{2c} - \frac{a}{2d}} + \xi_3 e^{-\frac{b}{2R}} \left(\frac{a}{c} > 0.8\right) \]

(Equation 6.2a)

\[ f\left(\frac{a}{c}, \frac{a}{d}, \frac{d}{R}, \frac{b}{c}\right) = \xi_1 \left[ e^{\frac{d}{2R} - \frac{a}{2d} - \frac{b}{2c} + 1} + \xi_2 \frac{d}{a} e^{\frac{b}{2c} - \frac{a}{2d}} + \xi_3 \frac{ab}{c} e^{-\frac{b}{2c}} \right] \left(\frac{a}{c} \leq 0.8\right) \]

(Equation 6.2b)

In Equation 6.2a and 6.2b, \( \xi_1, \xi_2, \xi_3 \) are constants in the influence coefficient functions, as summarised in Table 6-1.

| Table 6-1 Constants in influence coefficient functions (Equation 6.2) |
|-----------------|-----------------|-----------------|
| \( f(\frac{a}{c} > 0.8) \) | 0.3143 | 0.0570 | 0.4568 |
| \( f(\frac{a}{c} \leq 0.8) \) | 0.3296 | 0.1551 | 0.5538 |

6.3 Corrosion Model Development

To develop a statistical corrosion model considering the variability of soil properties and the random nature of pit growth behaviour, the raw corrosion data of the US NBS is thoroughly explored. A total of 208 sets of corrosion data that cover a wide range of soil properties in 74 locations of US are used to develop a statistical model for pit depth. To start with, the deterministic power low model \( a = k a^n \) was used for data fitting in each dataset, comprising a successive of corrosion measurements for buried cast iron pipes. With data fitting, the corrosion growth behaviour in each dataset is represented by a (unique) pair of pitting proportionality and exponent factors (i.e., \( k \) and \( n \)). Various distributions, including normal, lognormal, gamma, Weibull and generalised extreme value (GEV) distributions, are used for fitting the proportionality factors and exponent factors. The probability density functions that best fit the
proportionality and exponent factors were selected by the Kolmogorov-Smirnov and chi-squared test statistics, using EasyFit 5.6 (MathWave Technologies, 2016). It was found that the best fit for \( k \) factor was three parameters (3P) lognormal, followed by 3P Weibull and two parameters lognormal distribution. The distributions that fit \( n \) factor best were GEV, normal and 3P Weibull distributions in order. The statistics of fit are presented in Table 6-2 and Table 6-3 for \( k \) and \( n \), respectively. The parameters of distributions are determined by the maximum likelihood estimates method. The probability-probability plots are presented to compare the empirical cumulative distribution functions of \( k \) and \( n \) with the specified theoretical cumulative distribution functions, as shown in Figure 6-1. A good agreement between empirical and predicted cumulative distributions for both \( k \) and \( n \) was achieved.

**Table 6-2 Statistics of distribution fitting for \( k \) factor**

<table>
<thead>
<tr>
<th>Distribution</th>
<th>Parameters</th>
<th>Kolmogorov-Smirnov</th>
<th>Chi-Squared</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Statistic</td>
<td>Rank</td>
</tr>
<tr>
<td>Lognormal (3P)</td>
<td>( \sigma = 0.9870, \mu = 0.7110, \gamma = 0.0310 )</td>
<td>0.0735</td>
<td>1</td>
</tr>
<tr>
<td>Weibull (3P)</td>
<td>( \alpha = 1.036, \beta = 0.7574, \gamma = 0.0540 )</td>
<td>0.0781</td>
<td>2</td>
</tr>
<tr>
<td>Lognormal</td>
<td>( \sigma = 0.9044, \mu = -0.6179 )</td>
<td>0.0845</td>
<td>3</td>
</tr>
</tbody>
</table>

Note: 3P refers to three parameters.

**Table 6-3 Statistics of distribution fitting for \( n \) factor**

<table>
<thead>
<tr>
<th>Distribution</th>
<th>Parameters</th>
<th>Kolmogorov-Smirnov</th>
<th>Chi-Squared</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Statistic</td>
<td>Rank</td>
</tr>
<tr>
<td>GEV</td>
<td>( \xi = -0.2816, \sigma = 0.2919, \mu = 0.620 )</td>
<td>0.0363</td>
<td>1</td>
</tr>
<tr>
<td>Normal</td>
<td>( \sigma = 0.2903, \mu = 0.7236 )</td>
<td>0.0399</td>
<td>2</td>
</tr>
<tr>
<td>Weibull (3P)</td>
<td>( \alpha = 2.967, \beta = 0.8776, \gamma = -0.0594 )</td>
<td>0.0443</td>
<td>3</td>
</tr>
</tbody>
</table>

Note: GEV refers to generalised extreme value; 3P refers to three parameters.
Figure 6-1 Probability-probability plot for $k$ and $n$ factors

Note: Gen. Extreme Value refers to generalised extreme value; 3P refers to three parameters.

With the use of best fit distributions for the proportionality and exponent factors, a probabilistic corrosion model, based on the deterministic power law model ($a = kt^n$) can be obtained by representing $k$ and $n$ in a 3P lognormal and GEV distribution, respectively:
\[ a(t) = k t^n [k - \text{lognormal}(\mu, \sigma, \gamma), n - \text{GEV}(\mu, \sigma, \xi)] \]  \hspace{1cm} \text{(Equation 6.3)}

In Equation 6.3, \( a \) is pit depth (in mm); \( t \) is time (in years); \( \mu, \sigma, \gamma \) are location, scale and shape parameters of the lognormal distribution, respectively; and \( \mu, \sigma, \xi \) are location, scale and shape parameters of GEV distribution, respectively. In Equation 6.3, \( k \) and \( n \) are assumed statistically independent. However, the correlation analysis between \( k \) and \( n \) in the 209 datasets showed that the correlation coefficient \( r \) between \( k \) and \( n \) is as large as \(-0.55\), which is an unignorable dependence, according to Hinkle et al., (2003). To generate a random variable \( Y \) having a defined correlation \( r \) with another variable \( X \) that follows a distribution \( f \), \( Y \) can be represented as follows (Cramér, 2004):

\[ Y = rX + \sqrt{1 - r^2} X^* \]  \hspace{1cm} \text{(Equation 6.4)}

In Equation 6.4, \( X \) and \( X^* \) are independent variables following the same distribution, \( f \). Theoretically, it is equivalent to use either \( n \) or \( k \) to generate the other variable. Since there is a better fit observed for the exponent factor \( (n) \), compared with that of proportionality factor \( (k) \), as can be observed in Figure 6-2, the exponent factor \( (n) \) is used to generate \( k \) to reduce the error of approximation. Therefore, the corrosion model considering the correlation between the parameters is expressed as follows:

\[ a(t) = \left( m + \sqrt{1 - r^2} n^* \right)^n \cdot \left[ n^* - \text{GEV}(\mu, \sigma, \xi) \right] \]  \hspace{1cm} \text{(Equation 6.5)}

To verify the proposed models, the corrosion data at 1, 5, 10 and 30 years were computed and compared with the experimental data obtained from the NBS database, which is shown in Figure 6-2. The experimental data are represented by grey shaded histograms, while the predicted results are represented as lines. The corrosion model considering the correlation between \( k \) and \( n \) factors is indicated as model A, while the model that treats \( k \) and \( n \) as independent is indicated as model B. A good agreement between experimental data and model simulated data is observed, implying that both proposed models can reproduce the evolution of the pit depth for a relatively short exposure (less than 10 years). For a longer exposure (e.g., 30 years), the corrosion model A approximates the upper tail of the data more accurately than model B. Since the upper tail of the data greatly affects the accuracy of reliability predictions, model
A (i.e., Equation 6.5) is more appropriate in the prediction of corrosion pit depth in buried cast iron pipes.
Figure 6-2 Comparison of experimental data and models predicted results for (a) $t = 1$ year, (b) $t = 5$ years, (c) $t = 10$ years and (d) $t = 30$ years
6.4 Probability of Failure

According to fracture mechanics principles, a pipe is expected to fracture when the SIF exceeds its fracture toughness. This failure criterion, based on the theory of reliability, can be written in the form of a limit state function:

\[ G(K, K_c, t) = K(t) - K_c(t) \]  
(Equation 6.6)

In Equation 6.6., \( K(t) \) is the SIF (load effect) at time, \( t \); \( K_c \) is fracture toughness (a critical limit for the SIF). Using the limit state function, the probability of pipe failure, due to corrosion-induced cracking, can be determined:

\[ p_f(t) = P [G(K, K_c, t) \leq 0] = P [K(t) \geq K_c(t)] \]  
(Equation 6.7)

In Equation 6.7, \( P \) represents the probability of pipe fracture failure. Equation 6.7 shows an upcrossing issue, and it can be solved by use of the time-dependent reliability method (Melchers, 1999). Generally, the either the loading effect or the resistance is modelled by a stochastic process in the time-dependent reliability method. Since the pipe failure relies on the the time that elapses before the occurrence of the loading process, \( K(t) \), upcrossing a critical value (e.g., threshold), \( K_c(t) \) for the first time. The probability of the first upcrossing of the loading process to its threshold is called ‘first passage probability’, which can be determined as follows (Ditlevsen, 1983; Melchers, 1999):

\[ p_f(t) = 1 - [1 - p_f(0)] e^{-\int_0^t \nu d\tau} \]  
(Equation 6.8)

In Equation 6.8, \( p_f(0) \) denotes the probability of failure at time \( t = 0 \) and \( \nu \) denotes the mean rate for the loading process, \( K(t) \), to upcross the critical value, \( K_c(t) \). In most cases, the mean rate (\( \nu \)) of upcrossing is assumed to be zero and therefore Equation 6.8 can be expressed as follows:

\[ p_f(t) = p_f(0) + \int_0^t \nu d\tau \]  
(Equation 6.9)
The upcrossing rate ($\nu$) can be calculated through the Rice’s formula (e.g., Melchers, 1999):

$$
\nu = \nu^*_{Kc} = \int_{K_c}^{\infty} (\hat{K} - \hat{K}_c) f_{KK}(K_c, \hat{K}) d\hat{K}
$$

(Equation 6.10)

In Equation 6.10, $\nu^*_{Kc}$ denotes the upcrossing rate of the loading process, $K(t)$, corresponding to the critical value, $K_c(t)$. Further, $\dot{K}$ denotes the time derivative of $K$. Also, $\dot{K}_c$ denotes the derivative of $K_c$ with respect to time and $f_{KK}$ denotes the joint probability function of $K$ and $\dot{K}$.

To apply the first passage probability in reliability analysis, it would be ideal to develop an analytical closed-form solution to the Equation 6.10. However, driving such a solution is especially difficult; only limited work has been completed by researchers who have modelled the loading effect as a Gaussian process. For example, Ditlevsen (1983) investigated the upcrossing problem for a strictiral system that is modelled as a Gaussian process relative to the barrier levels. Li and Melchers (1993) derived an closed-form analytical solution for the upcrossing rate of a nonstationary Gaussian process and the solution has been applied in many engineering problems (e.g., Li & Mahmoodian, 2013; Li & Melchers, 2005; Yang et al., 2018). The Gaussian process, stationary or nonstationary, has the symmetric feature with zero skewness Gaussian distribution, indicating that there would be some realisations having negative values. It is known that most practical processes (e.g., wind speeds or pipe pressures) are essentially positive. To eliminate the unrealistic negative values in the realisation of a Gaussian process, a non-Gaussian process (e.g., lognormal process) can be used to represent the stochastic process of loading effect. Li et al. (2016) derived a closed-form solution of the upcrossing rate for a nonstationary lognormal process as follows:

$$
\nu^*_{Kc} = \frac{e^{\hat{e}_{Kc}(t)}}{K_c(t)\sigma(t)} \phi \left( \ln[K_c(t)] - \frac{\lambda(t)}{\sigma(t)} \right) \\
\times \left\{ e^\left[ \sqrt{1 - \Phi \left(1 - \frac{\ln[\hat{K}_c(t)] - \lambda_{\hat{K}K}(t)}{\sigma_{\hat{K}K}(t)} \right)} \right] - \Phi \left( \frac{\ln[\hat{K}_c(t)] - \lambda_{\hat{K}K}(t)}{\sigma_{\hat{K}K}(t)} \right) \right\}
$$

(Equation 6.11)
In Equation 6.11, $\varphi()$ is the probability density function of a standard normal variable and $\Phi()$ is the probability function of a standard normal variable. Further, $|$ denotes the condition. The parameters $\varepsilon(t)$ and $\lambda(t)$ for the lognormal random variable, $K(t)$, are calculated as follows (Papoulis, 1965):

$$
\varepsilon(t) = \sqrt{\ln(\frac{\sigma_k^2}{\mu_k^2} + 1)} \quad \text{(Equation 6.12a)}
$$

$$
\lambda(t) = \ln \left( \frac{\mu_k^2}{\sigma_k^2 + \mu_k^2} \right) \quad \text{(Equation 6.12b)}
$$

All other variables in Equation 6.11 can be determined according to the theory of stochastic processes (see Li & Melchers, 2005; Papoulis, 1965) as follows:

$$
\mu_{k|K} = E[K \mid K = K_c] = \mu_k + \rho \frac{\sigma_k}{\sigma_K} (K_c - \mu_k) \quad \text{(Equation 6.13a)}
$$

$$
\sigma_{k|K}^2 = \left( \sigma_K^2 (1 - \rho_k^2) \right)^{1/2} \quad \text{(Equation 6.13b)}
$$

For Equation 6.13a and 6.13b, the variable can be expressed as follows:

$$
\mu_k = \frac{d \mu_k(t)}{dt} \quad \text{(Equation 6.13c)}
$$

$$
\sigma_k = \left[ \frac{\partial^2 C_{kk} (t_i, t_j)}{\partial t_i \partial t_j} \right]_{i = j}^{1/2} \quad \text{(Equation 6.13d)}
$$

$$
\rho_k = \frac{C_{kk} (t_i, t_j)}{\left[ C_{kk} (t_i, t_j) C_{kk} (t_i, t_j) \right]^{1/2}} \quad \text{(Equation 6.13e)}
$$

The cross-covariance function can be expressed as follows:

$$
C_{kk} (t_i, t_j) = \frac{\partial C_{kk} (t_i, t_j)}{\partial t_j} \quad \text{(Equation 6.13f)}
$$
Based on the relationships in Equation 6.13, all the variables in Equation 6.10 can be determined.

### 6.4.1 Stochastic Model

Therefore, it can be observed that the load effect quantified by SIF is a time-dependent process with various factors involved (applied stress, geometries of pipes and corrosion pit growth etc.). To consider the random nature of the SIF, it is necessary to model the state of SIF as a stochastic process (i.e., a nonstationary lognormal process) as used in this study. According to Li and Melchers (2005), a random variable, \( \xi_k \), is introduced with the mean being one (i.e., \( E(\xi_k) = 1 \)) and \( \text{COV}, \lambda_k \), being constants. Therefore, the SIF is expressed as follows:

\[
K(t) = K_m(t) \cdot \xi_k \quad \text{(Equation 6.14)}
\]

In Equation 6.14, \( K_m(t) \) is a pure time function obtained from Equation 6.1. The statistics of \( K(t) \) can be determined by use of the technique, e.g., Monte Carlo simulation. The mean and autocovariance functions of \( K(t) \) are calculated as follows (Li & Melchers, 2005):

\[
\mu_K(t) = E[K(t)] = K_m(t) \cdot E[\xi_k] = K_m(t) \quad \text{(Equation 6.15a)}
\]

\[
C_{KK}(t_i, t_j) = \lambda_k^2 \rho K_m(t_i)K_m(t_j) \quad \text{(Equation 6.15b)}
\]

In Equation 6.15, \( \rho \) denotes (auto)correlation coefficient for \( K(t) \) between time \( t_i \) and \( t_j \).

### 6.4.2 Sensitivity Analysis

Statistical data is significant in the assessment of pipe failure probability. Because there is a lack of full statistical information on some parameters, it is vital to identify the contribution of each random variable to the probability of failure so that more attention can be taken to the most influential factors, which can be reached using a probability sensitivity index. Since the Equation 6.6 is a highly nonlinear limit state
function, the Hasofer-Lind reliability index is used in this study. The probability sensitivity index \( \alpha_i \) can be represented as follows (Nowak & Collins, 2012):

\[
\alpha_i = \frac{-\frac{\partial G}{\partial Z_i}}{\sqrt{\sum_{k=1}^{n} \left(\frac{\partial G}{\partial Z_k}\right)^2}} \quad \text{(Equation 6.16)}
\]

In Equation 6.16, the variables can be expressed as follows:

\[
\frac{\partial G}{\partial Z_i} = \frac{\partial G}{\partial X_i} \frac{\partial X_i}{\partial Z_i} = \frac{\partial G}{\partial X_i} \sigma_{X_i} \text{MPa} \sqrt{\mu_{X_i} K_{IC}(t)} \quad \text{(Equation 6.17a)}
\]

\[
\sum_{i=1}^{n} (\alpha_i)^2 = 1 \quad \text{(Equation 6.17b)}
\]

\[
Z'_i = \beta \alpha_i \quad \text{(Equation 6.17c)}
\]

\[
G(Z'_1, Z'_2, \ldots, Z'_n) \quad \text{(Equation 6.17d)}
\]

\[
Z_i = \frac{X_i - \mu_{X_i}}{\sigma_{X_i}} \quad \text{(Equation 6.17e)}
\]

In Equation 6.17, \( X_i \) is the random variable, \( n \) is the number of design variables, \( \sigma_{X_i} \) is the standard deviation of the random variable \( X_i \), \( \mu_{X_i} \) is the mean of the random variable, \( X_i \) and \( \beta \) is the shortest distance between the origin of ‘standard form’ variables \( (Z_i) \) and the limit state function. As such, \( (Z'_1, Z'_2, \ldots, Z'_n) \) is the design point (also known as checking point) on the surface of limit state function. To determine the \( 2n+1 \) unknowns (i.e., \( \alpha_i \), \( \beta \) and \( Z'_i \)), the iterative method is used and the procedures are presented as follows (Nowak & Collins, 2012):

1) Initialise the design point, \( \{x'_i\} \), for \( n-1 \) of the mean values of random variables \( X_i \). Solve the limit state function, \( G=0 \), for the rest of random variables.

2) Determine the reduced variates, \( \{z'_i\} \), by the following equation:
3) Calculate the partial derivatives of the $G_i$ with respect to $\{z_i^*\}$ the following:

$$G = \begin{bmatrix} G_1 \\ G_2 \\ \vdots \\ G_n \end{bmatrix}$$  \hspace{1cm} (Equation 6.19)

In Equation 6.19, $G$ can be calculated as follows:

$$G_i = -\frac{\partial g}{\partial Z_i} \text{ evaluated at design point}$$  \hspace{1cm} (Equation 6.20)

4) Estimate the shortest distance, $\beta$, by the following equation:

$$\beta = \frac{G \cdot z^*}{\sqrt{G^\top \cdot G}}$$  \hspace{1cm} (Equation 6.21)

In Equation 6.21, $z$ can be calculated as follows:

$$z^* = \begin{bmatrix} z_1^* \\ z_2^* \\ \vdots \\ z_n^* \end{bmatrix}$$  \hspace{1cm} (Equation 6.22)

5) Determine a vector of Hasofer-Lind reliability indexes by the following equation:

$$\begin{bmatrix} \alpha_1 \\ \alpha_2 \\ \vdots \\ \alpha_n \end{bmatrix} = \frac{G}{\sqrt{G^\top \cdot G}}$$  \hspace{1cm} (Equation 6.23)

6) Determine a new design point for $n-1$ of the variates, $\{z_i^*\}$, and calculate the original variates according to Equation 6.18 in Step 2, as follows:
\[ z_i^* = \alpha_i \beta \]  
(Equation 6.24)

\[ x_i^* = \mu_i + z_i^* \sigma_i \]  
(Equation 6.25)

7) Repeat the previous steps until the convergence is reached.

### 6.5 Worked Example

To illustrate the proposed method for predicting the failure probability of corroded pipes, a case study of a cast iron pipe with a sharp corrosion pit is conducted, as illustrated in Figure 6-3. A pipe, with the radius of 230.13 mm and wall thickness of 16 mm, is considered. For simplicity, the aspect ratio is assumed to be 1 (i.e., \( a/c = 1.0 \)) and the ratio of pit width to half-length (i.e., \( b/c \)) is assumed as 0.5. Some basic parameters are assumed to have deterministic values. That is, Poisson’s ratios and elastic modulus for pipe are set to be 0.23 and 130 GPa (Coffin, 1950), respectively. However, corrosion depth, pipe radius, wall thickness and internal pressure are considered random, as presented in Table 6-4. The fracture toughness values range from 17 to 24.3 MPa\( \sqrt{m} \), according to the testing results of Mohebbi et al., (2010). The corrosion growth is predicted by Equation 6.5 (i.e., Model A), with the corrosion parameters shown in Table 6-3. Figure 6-4 shows the distributions of pit depths predicted by the proposed corrosion model for 10, 30 and 50 years, showing the temporal evolution of corrosion depth distributions. This makes sense since as exposure time increases, the probability of finding a deeper corrosion increases.

![Figure 6-3 A pressurised pipe with an external sharp corrosion pit](image-url)
The Monte Carlo algorithm has been adopted as a pragmatic approach to determine the statistics of \( K_i(t) \), as illustrated in Figure 6-5. It starts with generating a vector of the basic random variables, \( \{a'_i, p'_i, d'_i, D'_i, \ldots\} \), in the \( j^{th} \) Monte Carlo step substituted into the Equation 6.1 and 6.2 for the \( i^{th} \) time evaluation. At each time point, the Monte Carlo simulation repeats 5,000 times to assure the convergence of the Monte Carlo estimates, which constitutes an unbiased estimate of \( K_i(t) \), as per Caleyo et al. (2009). The mean function \( (\mu_{k_i}) \) and standard deviation \( (\sigma_{k_i}) \) of load effect, \( K_i(t) \), can be calculated as a function of time, \( t \). Then the upcrossing rate, \( \nu \),
can be obtained from Equation 6.10 for a given autocorrelation coefficient, $\rho$, followed by the calculation of the probability of pipe failure, $p_f$, using Equation 6.9.

Figure 6-5 Flow chart of the Monte Carlo simulation

Figure 6-6 shows the results of pipe failure probability versus exposure time, with three autocorrelations ($\rho$) of the SIF. It can be seen that when the service life increases, the corresponding probability of fracture failure in pipe increases. It can also be observed that $\rho$ has a significant impact on the probability of failure, justifying the necessity of incorporating both the stochastic process theory and first passage probability concept into the reliability assessment.
Figure 6-6 Probability of pipe fracture failure for different coefficients of correlation ($\rho$)

The probabilities of fracture failure with respect to different fracture toughness values are presented in Figure 6-7. It can be observed that different fracture toughness values can result in different probabilities for failure, and a larger fracture toughness ($K_{IC}$) lead to a smaller probability of pipe fracture failure at a given time point.

Figure 6-7 Probability of pipe fracture failure for different fracture toughness values
By use of the statistical data that is presented in Table 6-3 and Table 6-4, the probability sensitivity indexes of four basic random variables (e.g., corrosion depth, pipe radius, wall thickness and internal pressure) are determined by Equation 6.16 for different time, as presented in Figure 6-8. It can be seen that all variables have positive values except for pipe thickness. This makes sense that the increase of corrosion depth, pipe radius and internal pressure can lead to the increase of pipe failure probability, while the increase of wall thickness can contribute to the decrease of pipe failure probability. It can be observed from Figure 6-8 that the internal pressure ($p$) has the greatest contribution to the probability of pipe failure at the beginning of pipe service life, while its contribution consistently decreases during the rest of time. Similar trends can be observed in pipe wall thickness ($d$) and pipe diameter ($D$). In comparison, the probability sensitivity index of corrosion depth ($a$) increases continually and after 20 years of pipe age, it has the most influence on the probability of failure. This information has practical meaning that more attention can be paid to the most influential factors during the inspection and maintenance for aged pipes. The great effect of corrosion depth, as shown in Figure 6-8, again justifies the necessity of determining accurate values for corrosion depth during failure probability assessments of corroded pipelines.

![Figure 6-8 Change of probability sensitivity index with time](image_url)
6.6 Summary

A time-dependent reliability method for the assessment of fracture failure in corrosion-affected cast iron pipes has been proposed in this chapter. The concept of fracture mechanics is applied to establish a failure criterion for pressurised pipes with an external sharp corrosion pit. A probabilistic corrosion model that considers the correlation between parameters in the model has been developed. The SIF of pipe is modelled as a nonstationary lognormal process and the upcrossing method is employed to determine the probability of failure. An example is carried out to demonstrate the application of the proposed method in predicting the failure probability of corroded pipelines. The developed corrosion model, with consideration of statistical correlation between the model parameters (i.e., \( k \) and \( n \)), produced an accurate prediction of corrosion growth behaviour. The autocorrelation of the SIF process between two points in time has an important influence on the probability of failure, justifying the necessity of using the upcrossing method in pipe reliability assessments. The sensitivity analysis further shows that initially, internal pressure has the greatest contribution to the probability of pipe failure; however, the magnitude of the contribution of corrosion depth \( (a) \) consistently increases and becomes the dominating variable for pipe failure after 20 years of pipe age. It can be concluded that the proposed method can serve as a useful tool for engineers to predict the failures of corroded cast iron pipes with improved accuracy, enabling better risk management.
Chapter 7: Conclusions and Recommendations for Future Work

7.1 Conclusions

The main objective of this research is to develop a new method that allows accurate prediction of the remaining safe life of metal pipes buried in soil. The work includes laboratory corrosion tests, experimental tests and numerical simulations of corrosion effects on pipes, statistical analysis of corrosion-influencing factors in soils and pipe failure assessment. While much research has already been conducted on pipe failure analysis, the occurrences of pipe failure events have not been effectively prevented. As such, the current understanding of failures in buried pipes due to pitting corrosion is still limited. Most of the reported research works are based on small-scale immersion tests in relatively short exposure times, rather than burying pipes in real soils. The corrosion pits are often assumed to be blunt geometries, rather than more dangerous corrosion pits with sharp tips. Further, the application of fracture mechanics in pipe failure analysis and assessment is relatively limited. With these considerations in mind, an experimental program that is capable of producing corrosion data on full-size pipe sections in real soil and investigating corrosion effects on pipe mechanical properties was developed. Numerical simulations were conducted to investigate the effects of sharp corrosion pits on cast iron pipes based on fracture mechanics. The upcrossing method was employed in failure probabilities prediction by modelling the SIF of corroded pipes with a nonstationary lognormal process. This research provides fundamental insight into the corrosion behaviour of buried cast iron pipes and its corrosion-influencing factors in soils. It also contributes to the understanding of material deterioration and pipe failure mechanisms, potentially providing engineering guidelines for the water, oil and gas industries.

Based on the research presented in this thesis, the following main conclusions can be drawn.

1) The developed testing program produced effective corrosion data of cast iron pipes buried in realistic soil under laboratory conditions. It has been found that the corrosion rates of pipes are generally high at the initial exposure stage.
(e.g., \(i_{\text{corr}} > 0.02\text{mA/cm}^2\) for pipes buried in soil with \(\text{pH} = 2.5\)), while reducing and stabilising at a small value over time (e.g., \(i_{\text{corr}} \sim 0.01\text{mA/cm}^2\) after approximately 250 days). It has also been found that the mass loss measurements indicated a reduction of corrosion rates over exposure time, which is in reasonable agreement with the findings reported in the literature.

2) From the mechanical test results on corroded pipes, it has been found that both the fracture toughness and modulus of rupture indicated a decreasing trend versus corrosion time. A relationship between the reduction of fracture toughness and mass loss has been established. It has also been found from the microstructure analyses that the localised corrosion and formation of graphitised zones are primary causes for the degradation of the mechanical properties of cast iron pipes.

3) In the numerical simulation study, a new geometrical model for sharp corrosion pit was proposed and a series of three-dimensional pipes models were built to calculate the SIFs of pressurised pipes with various dimensions for the pipes and corrosion pits. It has been found that the maximum SIF of pipes subjected to high aspect ratio pits (e.g., \(a/c = 2.0\)) occurs at the surface point, while pipes subjected to low aspect ratio pits (e.g., \(a/c = 0.4\)) normally occurs at the deepest point. It also has been found that the pit width \((b)\) has a considerable effect on the SIF of pipes.

4) Based on the parametric FE studies, an expression of the maximum SIF of pipes has been developed considering the applied stress, pipe dimensions and corrosion pit dimensions (i.e., depth \(a\), width \(b\) and half-length \(c\)). It has been found that assuming corrosion pits are surface cracks will cause inaccurate estimates of risk.

5) In the statistical analysis of the NBS database, the corrosion data in each sample was characterised by two time-independent parameters (i.e., \(k\) and \(n\)) based on the power law model. It has been found that \(n\) is closely associated with the level of soil aeration. Grouping corrosion data, based on soil aeration, produces stronger correlations between soil properties and corrosion rates, compared with considering all soil samples as a whole. The correlation
analyses also show that most soil properties are closely related (e.g., soil resistivity and total salt content, and air-pore space and moisture equivalent).

6) Based on the NBS field data, a probabilistic corrosion model has been developed by modelling the exponent factor $n$ of the power law model as a GEV distribution and considering the statistical correlation between the factors $k$ and $n$. A good agreement between the field data and the model predictions has been observed. It has also been found that the corrosion model, considering the correlation between $k$ and $n$, produced a better approximation of the upper tail of the field data after 30 years of pipe age.

7) In the assessment of the probabilities of pipe facture failure, a time-dependent reliability method (i.e., the upcrossing method) was employed. The SIF of pipe was modelled as a nonstationary lognormal process and an example was undertaken to illustrate the application of the proposed method. It has been found that the correlation of the SIF process between two points in time had a significant impact on the probability of failure, justifying the necessity of using a time-dependent reliability method. It has also been found that the corrosion depth and internal pressure have the most influence on the probability of failure, compared with other parameters.

7.2 Recommendations for Future Work

Research is open ended. There are always new topics to pursue and room for improvement for existing research. In the context of this thesis, the following additional work can be undertaken in future research.

1) In the present research work, only cast iron pipe was used for the corrosion and mechanical tests. Evidently, the composition and microstructure of pipe materials can affect its corrosion behaviour and deterioration process. Pipes made by other materials, such as ductile iron and steel, should also be studied. Further investigations also need to be conducted to compare the effects of corrosion on material microstructure between different pipe materials.

2) In the corrosion tests, the soil has been oven dried before use to eliminate MIC. However, the presence of specific microorganisms in soil such as SRB
can accelerate corrosion and greatly affect the degradation of mechanical properties of buried pipes. This important factor needs to be considered in future work to improve understanding of corrosion mechanisms in buried pipes.

3) In the current FE analysis, only the opening deformation mode (Mode I) was considered. In actual service conditions, other deformation modes of fracture can also be found and the SIFs of pipe subjected to corrosion pits under mixed modes needs to be investigated. Pipes made of the ductile materials (e.g., ductile iron and steel) should be considered with the application of elastic-plastic fracture mechanics.

4) The corrosion model in the current work was basically developed by determining the statistics of proportionality and exponent factors ($k$ and $n$). It would be ideal to incorporate the key corrosion-influencing factors (e.g., saturation, pH and soil resistivity) into the corrosion model. Further research is needed in terms of conducting extensive corrosion experiments and mathematical modelling.

5) In the pipe failure assessment, the SIF of pipe was modelled as a lognormal process. However, in reliability analysis, the stochastic process can be of different distributions (e.g., gamma and chi-square distributions). Further research is required to derive the corresponding analytical solutions to Equation 6.9 or the so-called first passage probability.
References


England: Department of Transport, Transport and Road Research Laboratory. https://trl.co.uk/sites/default/files/CR266.pdf


172


https://doi.org/10.1111/j.1747-1567.2006.00134.x


Talbot, A. N. (1926). Strength properties of cast iron pipe made by different processes as found by tests. *Journal American Water Works Association*, 16(1), 1–44.


https://doi.org/10.1016/j.ibiod.2014.05.007


https://doi.org/10.1097/00010694-193109000-00003


