Pipeline Decommissioning Research Program

FINAL REPORT

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Disclaim

This research was supported by Enbridge Pipelines Inc. (Enbridge), which agreed that the research results can be shared with the Canadian Association of Energy and Pipeline Landowners Associations (CAEPLA).

The research was conducted to help understand the long-term corrosion progression, degradation of structural integrity and ground subsidence for decommissioning of Line 3 pipeline in place. The research outcomes would help predict the potential risks associated with the decommissioned pipelines.

This document was prepared for the exclusive use of Enbridge. The material in it reflects the judgment of the University of Calgary’s research team based on the information available at the time of document preparation. The judgement was based on limited data, special cases, and embedded assumptions.
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1. Introduction

Pipelines are prominent in debates on appropriate balances between economic, environmental and social goals. In addition to the safety concerns and potential impact on the environment, a critical issue raised in regulatory hearings is decommissioning of pipelines at the end of their service life. When pipelines are decommissioned and left in place, corrosion can occur both externally and internally. In the absence of cathodic protection (CP), the pipe steel suffers from general corrosion as well as localized corrosion at coating breakages, under disbonded coating, under solid deposit, etc. When the CP is applied, corrosion can still occur at the bottom of coating breakages and the coating disbondment due to shielding effect on the CP current. As a result, pipeline perforations caused by localized corrosion will be inevitable during the long-term decommissioning. Moreover, the microbial activity in soils can cause microbiologically-influenced corrosion (MIC) of the pipelines, resulting in localized pitting corrosion. Pipeline corrosion is also expected to be accelerated by loading from various sources such as surface load, soil load, etc.

Structural integrity of a decommissioned pipeline will be compromised due to sufficient degradation by both external and internal corrosion, and external loading such as surface load transfer. Loss of structural integrity could cause subsequent collapse over time, eventually leading to a measurable amount of ground subsidence. The risks associated with loss of structural integrity and ground subsidence depend on a number of environmental conditions and factors, including soil chemistry, soil saturation, depth of cover, orientation and extent of corrosion, surface loads, coating performance, CP level (if applicable), pipeline characteristics, levels of pipeline collapse, etc. All of these conditions and factors are interrelated, and thus, affect the potential risk and associated consequences.

Enbridge proposed to decommission in place 1,067 km of the existing Line 3 pipeline from Hardisty Terminal, Alberta to Gretna Terminal, Manitoba. During development of the application to the National Energy Board (NEB), it was identified that there was limited scientific knowledge base developed around pipelines in decommissioning. Moreover, an agreement was reached between CAEPLA and Enbridge to initiate a formal research program to study the long-term hazards associated with leaving a decommissioned pipeline in place. In March 2016, Enbridge commissioned the University of Calgary’s research team to work on the research program.
2. Theme I: Long-Term Corrosion Progression

The theme of long-term corrosion progression includes five interrelated research topics, all of which are essential to understanding of the fundamental and practical aspects of corrosion progression on decommissioned pipelines in the soil. At the same time, the research outcomes in this theme would provide data (i.e., the long-term corrosion rate of pipeline steel in the soil) to the theme of Structural Integrity for modeling of structural degradation of corroded pipelines over the decommissioning process.

2.1. Permeability of Cathodic Protection (CP) Current to Polyethylene Coating and the CP Current Demand for Pipeline Corrosion Protection at Coating Failures

2.1.1. Research background

Pipeline systems are normally protected from external corrosion attacks by a combination of coatings and CP. When pipelines are decommissioned and left in place, corrosion would occur at the bottom of coating breakages and under disbonded coating even if the coated pipeline is under CP, as the CP current can become shielded from reaching these areas [Cheng, 2013a; Kuang and Cheng, 2015a; Kuang and Cheng, 2015b]. The Line 3 pipeline is coated with polyethylene (PE) coating. However, PE has been recognized as the one linking to the CP shielding issue [Cheng, and Norsworthy, 2017; Thompson and Saithala, 2016]. Due to its high electrical strength, PE-based coatings are not expected to pass the CP current and thus shield the CP from reaching the pipe steel [Cheng, 2013b]. Thus, one of the key objectives of the research program is to investigate the shielding effect of various failure modes of PE coating occurring in the field on the CP current permeation.

At the same time, the CP current demand, if it is maintained for decommissioned pipelines, is expected to increase for corrosion protection during long-term decommissioning. CP shielding is still possible even if the coating does not contain defects due to the non-polar molecular structure of the coating. In this research, the CP current demands for corrosion protection of the pipeline under disbonded, defect-free PE coating are determined quantitatively.
The research results would provide the pipeline operator advice whether CP is required to apply on decommissioned Line 3 pipeline for effective corrosion protection at coating failures and thus an extended time period to maintain the structural integrity, as well as the quantitative recommendation of the CP current demanded for the purpose.

2.1.2. Research methods

PE membranes used in this work were supplied by Enbridge Pipelines. Steel specimens were cut from an X52 steel pipe, with a chemical composition (wt. %): 0.24 C, 1.4 Mn, 0.45 Si, 0.025 P, 0.015 S, 0.10 V, 0.05 Nb, 0.04 Ti and Fe balance. A simulated soil solution with the identical composition to the extracted solution from a clay soil in Regina, Canada was used in this work. To analyze the extracted soil solution, the international standard ISO 11048 was followed. The simulated soil solution was prepared by analytic grade chemicals and deionized water, including (g/L) 0.0755 NaHCO₃, 0.0092 NaCl, 0.0014 NaNO₃, 0.0773 Na₂SO₄, 0.0619 K₂SO₄, 1.116 CaSO₄·2H₂O and 0.662 MgSO₄·7H₂O. The solution pH was 6.0 to 6.5. All tests were conducted at ambient temperature (~22°C).

Fig. I-1. Schematic diagram of the self-made test rigs to measure the CP permeability of (a) disbonded defect-free PE coating (with varied thicknesses), and (b) a disbonded PE coating (2.0 mm in thickness) with a holiday with varied sizes.

Two self-designed test rigs enabling measurements of the CP permeability of the PE coating in the absence and presence of a holiday are shown in Fig. I-1. In Fig. I-1a, two testing chambers were separated by a defect-free PE coating membrane with varied thicknesses, and in Fig. I-1b, a PE
coating membrane with 2.0 mm in thickness contained a holiday with varied sizes. Three coating thicknesses, i.e., 1.0 mm, 1.5 mm and 2.0 mm, were selected to determine the effect of the coating thickness on CP permeability of the PE coating in Fig. I-1a. Five holiday diameters, i.e., 10.0 mm, 4.0 mm, 2.0 mm, 1.0 mm and 0.5 mm, were selected to investigate the effect of the holiday size on CP permeability of the coating containing a holiday in Fig. I-1b. In the two test rigs, a gap of 5 mm between the coating membrane and the steel electrode simulated the disbonding crevice under the coating.

![Diagram](image)

**Fig. I-2.** Schematic diagrams of the experimental setup to measure the CP permeability under disbonded PE coating in the simulated soil solution (a-front view, b-top view).

Fig. I-2 shows the experimental setup to measure the CP permeability at varied disbonding depths from the open holiday under disbonded PE coating in the simulated soil solution. To prepare an artificial disbondment, the PE coating (2.0 mm in thickness) was applied on adhesion zones of a Perspex sheet using a double-sided self-adhesive tape. The gap between the coating and the steel was determined by the thickness of the tape. The thickness of the created gap was defined as the disbonding thickness. A hole (10.0 mm in diameter) was opened on the coating to simulate an open holiday. Six potential/pH micro-probes were installed at the distances of 30 mm, 60 mm, 90 mm, 120 mm, 150 mm and 180 mm from the defect. The distance of the probing position to the open holiday was defined as the disbonding depth. To further investigate the effect of disbonding thickness on CP permeability at varied disbonding depths from the open holiday under disbonded
coating, the tape with three known thicknesses, i.e., 120 μm, 240 μm and 360 μm, was layered to establish the desired disbonding thicknesses.

2.1.3. **CP permeability of defect-free PE coating membranes with various thicknesses**

Fig. I-3 shows the optical views of the steel specimens after 30 days of testing in the simulated soil solution using the setup in Fig. 1a, where the PE coating membranes with various thicknesses are used and a CP potential of -925 mV (CSE) is applied. It is seen that, while the applied CP can fully protect the steel from corrosion, the steel under the disbonded PE coating corrodes after 30 days of testing in the soil solution even the CP is applied. Obviously, the applied CP does not provide protection to the steel under the PE coating. Moreover, the coating thickness does not affect the steel corrosion visually.

Fig. I-3. Optical views of the steel after 30 days of testing in the simulated soil solution which is separated by a PE coating membrane with varied thicknesses (a – no coating, b – 1.0 mm, c – 1.5 mm, d – 2.0 mm) in Fig. 1a, while a CP potential of -925 mV (CSE) is applied.

Fig. I-4 shows the CP current density measured on X52 steel electrode in the simulated soil solution separated by a PE coating membrane with varied thicknesses under the CP potential of -925 mV (CSE). It is seen, at individual coating thicknesses, the current density tends to increase negatively with time, indicating the increased CP current permeation through the coating. At individual
testing times, the cathodic current through the coating decreases slightly with the increasing coating thickness and the current density difference between each thickness becomes smaller with the coating thickness increases. For example, after 30 days of testing, the cathodic currents measured are about -0.020, -0.017 and -0.016 nA/cm² when the coating thicknesses are 1.00 mm, 1.50 mm and 2.00 mm, respectively. Thus, the CP current permeation is reduced as the coating thickness increases. It is further noticed that all the measured current densities are cathodic, indicating that the PE coating does not behave like an ideal capacitor [Kuang and Cheng, 2015a]. Although the CP current can permeate through the coating, the cathodic current is negligible (i.e., less than 0.3 nA/cm²), compared to the average polarization current density of -16.25 μA/cm² of a bare X52 steel under the CP potential of -925 mV (CSE) in the same soil solution, as shown in Fig. I-5.

Fig. I-4. CP current densities measured on X52 steel electrode in the simulated soil solution separated by a PE coating membrane with varied thicknesses of (a) 1.0 mm, (b) 1.5 mm and (c) 2.0 mm under the CP potential of -925 mV (CSE).
Fig. I-5. Polarization current density of X52 steel at the applied CP potential of -925 mV (CSE) in the simulated soil solution.

To further characterize the CP shielding by the disbonded PE coating, the potential of the steel in the simulated soil solution separated by the PE coating with various thicknesses as a function of time is measured using the setup in Fig. 1a, and the results are shown in Fig. I-6. It is seen that the potential of the steel is much less negative than the applied CP potential of -925 mV (CSE), indicating that the steel is shielded from the CP current by the PE coating membrane. With the increasing time, the potentials increase slightly and reach steady values of about -790 mV (CSE), which is approximately the corrosion potential of X52 pipeline steel in the simulated soil solution [Lins et al., 2012]. Thus, the CP current, even it is applied, would be shielded from reaching the steel by the PE coating for corrosion protection.

Fig. I-6. Potential of the steel in the simulated soil solution separated by the PE coating membrane with varied thicknesses as a function of time.
2.1.4. **CP permeability of PE coating containing a holiday with varied sizes**

Fig. I-7 shows the optical views of the steel specimen after 15 days of testing in the simulated soil solution separated by a PE coating membrane containing a holiday with varied sizes (i.e., setup in Fig. I-1b), where a CP potential of -925 mV (CSE) is applied at the holiday. It is seen that there is no obvious corrosion observed on the steel specimens when the coating holiday is sized 10 mm and 4 mm in diameter. This indicates that CP current can penetrate through the holiday and protect the steel from corrosion. However, when the holiday is 2 mm in diameter, the steel is corroded slightly, and as the holiday is as small as 1 mm and 0.5 mm in diameter, the steel suffers from corrosion apparently. Thus, the applied CP is partially or fully shielded from reaching the steel for corrosion protection. The results show that the CP permeation through a coating holiday depends on its size. In this work, when the holiday is as small as 2 mm in diameter, the CP current starts to be shielded.

Fig. I-7. Optical views of the steel specimens after 15 days of testing in the simulated soil solution separated by a PE coating membrane containing a holiday with varied sizes (a) 10 mm, (b) 4 mm, (c) 2 mm, (d) 1 mm, (e) 0.5 mm in Fig. 1b, while a CP potential of -925 mV (CSE) is applied at the holiday.
Fig. I-8 shows the cathodic current density measured on the steel in the simulated soil solution separated by the PE coating containing a holiday with varied sizes at the CP potential of -925 mV (CSE) as a function of time. It is seen that, initially, the cathodic current density of the steel is close to the required CP current density for a full CP (i.e., -16.25 μA/cm\(^2\) as seen in Fig. I-5) when the holiday diameter is 10 and 4 mm. This indicates that there is sufficient CP current permeating through the holiday to protect the steel. The cathodic current density drops rapidly and remains at a relatively stable value of around -2.7 μA/cm\(^2\), which is due to the formation of calcareous deposit on the steel surface. Whereas the cathodic current density is much lower than the required CP current density for coating holidays with the diameter of 2 mm, 1 mm and 0.5 mm, indicating the CP current is partially or fully blocked by the small holidays.

![Graph showing cathodic current density](image)

Fig. I-8. CP current density measured on the steel in the simulated soil solution separated by the PE coating containing a holiday with varied sizes at the CP potential of -925 mV (CSE) as a function of time.

Fig. I-9 shows the potential of the steel in the soil solution separated by a PE coating containing a holiday with varied sizes at the CP potential of -925 mV (CSE) as a function of time. It is seen that, with the increasing holiday size, the potential is more negative. The potential of the steel reaches the steady-state value of about -925 mV (CSE) (i.e., the applied CP potential) when the holiday diameter is 10 mm and 4 mm. Thus, the CP completely penetrates through the holidays. For the holidays with the diameter of 2 mm, 1 mm and 0.5 mm, the potential of the steel is less
negative and is steady at about -900, -850 and -820 mV (CSE), respectively. This demonstrates that the CP is shielded, at least partially, by the small holidays.

![Graph](image)

**Fig. I-9.** Potential of the steel in the soil solution separated by a PE coating containing a holiday with varied sizes at the CP potential of -925 mV (CSE) as a function of time.

2.1.5. **CP permeability at varied disbonding depths from the open holiday under PE coating disbondment**

Fig. I-10 shows the time dependence of cathodic current density of X52 steel under disbonded PE coating at varied disbonding depths from the open holiday (10 mm in diameter) with various disbonding thicknesses in the simulated soil solution, where a CP potential of -925 mV (CSE) is applied at the holiday. It is seen that the cathodic current density is close to the required CP current density for a full CP (i.e., -16.25 μA/cm² as seen in Fig. I-5) at the open holiday, while the value of current density under the disbonded PE coating is much less than that. This demonstrates that the CP is applied on the holiday, but is blocked by the coating disbondment. Moreover, at individual disbonding depth, the current density increases slightly with the increase in the disbonding thickness. This indicates that, although the coating disbondment still effectively shields the CP current from reaching the steel for corrosion protection, the increasing disbonding thickness favors the CP current permeation under coating disbondment.

Fig. I-11 shows the time dependence of the distribution of local potential under disbonded PE coating at varied disbonding depths from the open holiday (diameter in 10 mm) with various
disbonding thicknesses in the simulated soil solution, where a CP potential of -925 mV (CSE) is applied at the holiday. It is seen that, generally, the potential decreases rapidly after testing in the first few days, and then reach a relatively steady value. At the open holiday, the potential is the applied CP potential, therefore the CP completely reach the steel electrode under the open defect. While under the coating disbondment, the potential tends to be less negative and with increase in the disbonding depth, the potential shifts toward the positive direction. Identical to previous results, the CP shielding tends to be more significant as the disbonding depth increases. Moreover, at individual disbonding depth, the potential tends to be more negative with the increasing disbonding thickness. Thus, the permeability of CP current increases with the disbonding thickness becomes wider.

Fig. I-10. Time dependence of cathodic current density of X52 steel under disbonded PE coating at varied disbonding depths from the open holiday (10 mm in diameter) with various disbonding thicknesses of (a) 120 μm, (b) 240 μm and (c) 360 μm in the simulated soil solution, where a CP potential of -925 mV (CSE) is applied at the holiday.
2.1.6. **Permeability of PE coating to CP**

The permeability of the PE coating to CP current can be described by the ratio of the current density which passes through the coating to the required cathodic current density to protect the steel completely. This ratio is expressed by the equation below:

\[
p = \frac{i_{CP}}{i_0} \times 100\%
\]

where \(p\) is the permeability of coating, \(i_{CP}\) is the CP current density at a certain CP potential, and \(i_0\) is the CP current density for complete cathodic protection at the same CP potential. The calculated permeability of PE coatings with different thickness is shown in Table I-1. It is seen that the CP permeability of PE coating is quite low, indicating that almost all CP current can be...
blocked by PE coating. The coating thickness has little effect on the CP permeability of PE coating. Therefore, PE coating is impermeable to CP current.

Table I-1. Permeability of PE coatings with different thickness at the CP potential of -925 mV (CSE).

<table>
<thead>
<tr>
<th>Thickness (mm)</th>
<th>$i_0$ ($\mu$A/cm$^2$)</th>
<th>$i_{CP}$ (nA/cm$^2$)</th>
<th>$P$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>$-2.04 \times 10^{-2}$</td>
<td>$0.00013%$</td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td>$-16.25$</td>
<td>$-1.73 \times 10^{-2}$</td>
<td>$0.00011%$</td>
</tr>
<tr>
<td>2.0</td>
<td>$-1.61 \times 10^{-2}$</td>
<td>$0.00010%$</td>
<td></td>
</tr>
</tbody>
</table>

2.1.7. Summary

The disbonded PE coating is impermeable to CP current. Under the coating disbondment, the measured CP current density is too small to protect the pipeline steel due to the significant blocking effect of the PE coating, therefore corrosion of the steel under the disbonded coatings still occurs.

The geometrical factor of coating holiday plays an essential role in CP permeability of PE coating. With the increases in coating holiday size, the CP permeability of coating increases. For coating with holiday diameter larger than 4 mm, enough CP current can be penetrated through the holiday to protect the steel from corrosion attack.

Under the coating disbondment, the geometrical factor of the coating disbondment plays an essential role in CP permeability. With the increases of disbonding depth toward the disbondment bottom, the potential of the steel becomes positive, which is due to CP current diffusion can be shielded by coating disbondment and the formation of corrosion product. When the disbondment becomes wider, the CP shielding effect is mitigated.

2.1.8. Implication

For decommissioned Line 3 pipeline in place, it is expected that the external PE coating has degraded over the long-term service in the soil. The degradation modes may include disbonding, breakage, and/or disbonding from a holiday. From the present research results, the CP current, if
it is applied on the decommissioned Line 3, can be shielded from reaching the pipeline steel for corrosion protection at the coating failures as described above.

2.2. Uniform corrosion and microbiologically influenced corrosion (MIC) of pipeline in the soil

2.2.1. Research background

Pipelines decommissioned in place would experience corrosion. The external coatings have degraded over a long-term of service in the soil or will degrade during decommissioning. The CP may not be maintained. Even it is applied, the CP current can be shielded from reaching the pipe steel for corrosion protection at coating failures, as described above. Various factors relevant to pipeline decommissioning, such as soil chemistry, moisture content in the soil, dissolved oxygen, gassing condition, chloride ion concentration, temperature, etc., will affect the corrosion mechanism and rate of the pipeline [Cheng, 2013a]. In particular, the microorganism existing in the soil is believed to enhance the pipeline corrosion remarkably [Little and Lee, 1997].

Soil corrosion of steels is complex, and the complexity arises from numerous affecting factors and the difficulty to reproduce the corrosive environments encountered in the field. For buried pipelines, the pipe steel corrodes in either soil solution or wet soil with various moisture contents. Moreover, the thickness of the soil solution can be varied, depending on the water condensation occurring under different temperature-humidity combinations. While the presence of dissolved oxygen increases the steel corrosion, the absence of oxygen facilitates the culturing and growth of anaerobic bacteria such as sulfate-reducing bacteria (SRB), which would increase the corrosion rate remarkably. This research attempts to reproduce the soil corrosion environments in the laboratory to obtain testing results that are representative of the actual conditions.

The research outcomes in this topic provide mechanistic information about corrosion of decommissioned pipeline in the soil, and obtain corrosion rates which serve as the foundation to develop models to predict the long-term corrosion progression during decommissioning.

2.2.2. Research methods

The steel specimens and soil solution used are identical to those described in section 3.1.2. Weight-loss tests were conducted to measure corrosion rate of the steel under various conditions. After 14
days of immersion in the solution, the steel specimens were removed, and the corrosion products were cleaned using a pickling solution containing inhibitor imidazoline derivative (0.001 M). After rinsing with distilled water, cleaning in absolute ethanol and drying in high-purity N₂ stream, the specimens were weighed using an electronic balance with an accuracy 0.1 mg. The corrosion rate (CR) of the steel in mm/y was calculated by:

\[
CR = \frac{8.76 \times 10^4 \Delta m}{\rho At}
\]  

(2)

where \( \Delta m, \rho, A \) and \( t \) are weight loss (g), density (g/cm³) and area (cm²) of the specimens, and time (h), respectively.

For MIC testing, SRB (*Desulfovibrio desulfuricans*, ATCC 13541) isolated from the soil environment were used. The SRB culturing medium contained (g/L): \( \text{K}_2\text{HPO}_4 \) 0.5, \( (\text{NH}_4)_2\text{SO}_4 \) 0.5, \( (\text{NH})_2\text{Fe(SO}_4)_2 \) 0.2, \( \text{MgCl}_2 \) 0.3, sodium citrate 5, yeast extract 1.0 and sodium lactate 3.5 (pH 7.2). The SRB were incubated at 30 °C. Before inoculation, the SRB culturing medium was autoclaved at 121 °C for 20 min, and then deoxygenated by purging with high-purity N₂. The simulated soil solution with addition of 10 wt.% SRB culturing medium was used as the MIC test solution. To count the SRB cells attached on the steel specimens, the specimen surface was stained using a fluorescent dye (Molecular Probes™ FilmTracer™ LIVE/DEAD® Biofilm Viability Kit) in darkness for 10 min. The stained surface was imaged using a confocal laser scanning microscopy (CLSM).

2.2.3. **Effect of dissolved O₂ content on pipeline corrosion and MIC in simulated soil solution**

The role of the content of dissolved O₂ in soil solutions is dual. An elevated dissolved O₂ content would increase the steel corrosion via the enhanced electrochemical cathodic reaction. At the same time, the presence of dissolved O₂ inhibits the SRB growth, reducing the MIC rate of the steel. Thus, the dissolved O₂ is critical to the soil corrosion mechanism of buried pipelines, and to the accuracy and reliability of mechanistic model for long-term corrosion prediction.

The dissolved O₂ content in the simulated soil solution was controlled by different gassing conditions, i.e., purging with high-purity N₂, open to air, and purging with 5 % \( \text{CO}_2/\text{N}_2 \), which achieve the dissolved O₂ contents of 3.9 ppm, 7.7 ppm and 0.4 ppm, respectively.
Fig. I-12 shows the fluorescence images of live and dead SRB cells staining on the surface of the steel specimens after 14 days of testing in the soil solution under varied gassing conditions, where green and red colors indicate live and dead cells, respectively. It is seen that, under N₂-purging, there are more live SRB cells than dead cells, as seen in Figs. I-12a and I-12b. When the solution is open to air, the content of SRB cells, especially the live cells, decreases remarkably (Figs. I-12c and I-12d). However, the contents of live and dead SRB cells on the specimen surface increase in the presence of 5 % CO₂ (Figs. I-12e and I-12f). The results further confirm that CO₂ facilitates the SRB growth.

Fig. I-12. Fluorescence images of live and dead SRB cells on the specimen surface after 14 days of testing in the simulated soil solution under varied gassing conditions, where green and red colors indicate live and dead cells, respectively (a, b) N₂, (c, d) air, (e, f) 5 % CO₂.
The corrosion rate of the steel calculated from weight-loss measurements after 14 days of testing is shown in Fig. I-13. In the absence of SRB, the corrosion rate of the steel in the N₂-purging solution is the smallest, with the value of 0.12 mm/y, while there is the largest corrosion rate of 0.41 mm/y in the air-purging solution (Fig. I-13a). In the presence of SRB in the soil solution, the corrosion rates of the steel increase considerably under individual gassing conditions compared to the control specimen. The smallest corrosion rate of 0.72 mm/y is recorded in the presence of air, while the corrosion rate reaches the highest value of 2.15 mm/y in the CO₂-purging solution (Fig. I-13b). The corrosion rate of the steel in the N₂-purging solution is 1.46 mm/y, which is over 10 times of that measured in the absence of SRB. Thus, the presence of SRB accelerates the steel corrosion in the soil solution. The corrosion rates measured under varied gassing conditions are directly related to the SRB cell counts on the specimen surface.

Fig. I-13. Corrosion rates of the steel calculated from the weight-loss measurements after 14 days of testing in the simulated soil solution in the (a) absence and (b) presence of SRB, respectively.
Obviously, the anaerobic SRB accelerate steel corrosion over 10 times. Moreover, when the soil solution contains dissolved CO$_2$, there is a synergism of SRB and CO$_2$ on increased corrosion of pipeline steel. CO$_2$ is able to facilitate the SRB growth. Upon dissolution in water, the dissolved CO$_2$, as an electron donor, can be utilized by SRB to promote the growth of sessile SRB cells. As a result, more sessile SRB cells attach to the steel surface and participate in the corrosion processes. This is an important mechanism that the steel corrosion is enhanced in the co-existence of SRB and dissolved CO$_2$ in the environment.

2.2.4. **Effect of moisture content in soil on pipeline corrosion and MIC**

Compared to aqueous soil electrolytes, wet soils are the more common environment where pipeline corrosion occurs. It is well accepted that the moisture content in the soil is critical to corrosion reaction and its rate. Thus, tests were conducted on X52 pipeline steel in Regina clay soil containing varied water contents to simulate the moisture-soaked soil in the field.

Live/dead staining of adherent SRB cells on the steel specimen surface after 21 days of testing in the soil with varied water contents is shown in Fig. I-14, where green and red colors in the fluorescence images indicate live and dead SRB cells, respectively. It is seen that very few SRB cells are attached on the steel specimen when the soil contains a low water content (Fig. I-14a). As the water content increases, the counts of live and dead SRB cells increase (Figs. I-14b and I-14c). Thus, the increased water content in the soil promotes the SRB growth. Moreover, by comparison of Figs. I-14b and I-14d, the presence of CO$_2$ in the soil facilitates the growth of SRB at the identical water content, and there are more SRB cells attached to the specimen.

Fig. I-15 shows the corrosion rates of the steel obtained from weight-loss measurements after 21 days of testing in the soil with varied water contents. It is seen that the corrosion rate of the control specimen (i.e., no SRB) in the soil with a mass ratio of soil to water of 5:3 is low, with an average value of 0.05 mm/y only. When SRB are contained in the soil, the corrosion rate of the steel increases. Moreover, the corrosion rate increases with the increasing water content in the soil. When the mass ratios of soil to water are 5: 1, 5: 3 and 5: 5, the corrosion rates of the steel are 0.10 mm/y, 0.28 mm/y and 0.48 mm/y, respectively. The corrosion rate of the steel in the SRB-containing soil with the water content of 5: 5 is about ten times of that of the control specimen, indicating the increasing effect of SRB on pipeline corrosion in soils.
Fig. I-14. Live/dead staining of adherent SRB cells on the specimen surface after 21 days of testing in the soil with varied conditions, where green and red colors indicate live and dead cells, respectively. The mass ratio of soil and water is (a) 5:1, (b) 5:3, (c) 5:5, (d) 5:3 and CO$_2$ is present.

Fig. I-15. Corrosion rates of the steel obtained from weight-loss measurements after 21 days of testing in the soil with varied water contents. The ratios marked in the x-axis are the mass ratios of soil to water in the soil.
It has been accepted that the water content is one of the most important factors influencing the steel corrosion in soils [Yan et al., 2014]. In abiotic soils, the high water content can decrease the resistivity of the soil, resulting in a high corrosion rate. In the SRB-containing soil environment, the increase of the water content in the soil can favor the growth of SRB, where the increasing content of sessile SRB cells contributes to accelerated corrosion of the steel. At the same time, since the SRB can obtain electrons directly from Fe in soil with a limited organic carbon source, a higher water content in the soil makes the charge transfer more easily, thus accelerating the dissolution of the steel.

2.2.5. Effect of soil layer thickness on pipeline corrosion and MIC

Fig. I-16 shows the CLSM fluorescence images of the specimen surface under soil with various thicknesses after 21 days of testing, where the green and red colors indicate the live and dead SRB cells, respectively. It is seen from the color change that the number of SRB cells on the specimen decreases with the increasing thickness of the soil layer. For the bare steel specimen, live SRB cells are detected, with some dead cells found locally (Fig. I-16a). The ratio of the live to dead SRB cells, i.e., the area of the green color to that of red color, decreases with the increasing soil thickness. The result indicates that the presence of the soil layer affects adversely the growth of sessile SRB cells on the specimen, which is probably due to the blocking effect of the soil layer on transportation of nutrients towards the steel surface.

Fig. I-17 shows the corrosion rates of the steel determined from eight-loss measurements after 21 days of testing as a function of the soil layer thickness in the absence and presence of SRB in the soil environment. It is seen that, at individual soil layer thickness, the corrosion rate of the steel is over 10 times of that in the absence of SRB, indicating the accelerated corrosion by SRB. Generally, in the absence of SRB n, the corrosion rate increases gradually with the soil layer thickness (Fig. I-17a), indicating that the presence of the soil layer accelerates the steel corrosion. However, in SRB-containing soils, the corrosion rate of the steel increases to about 1.0 mm/y when the soil layer thickness is up to 3 mm. After that, the corrosion rate decreases with the soil layer thickness (Fig. I-17b). Thus, the presence of a soil layer and its thickness affects the MIC of pipeline steel.
Fig. I-16. CLSM fluorescence images of the steel specimen surface covered with various thicknesses of soil layer after 21 days of testing, where the green and red colors indicate the live and dead SRB cells, respectively (a) 0 mm, (b) 3 mm, (c) 5 mm, (d) 10 mm, (e) 15 mm, (f) 25 mm.
Fig. I-17. Corrosion rates of the steel determined from weight-loss measurements after 21 days of testing as a function of the soil layer thickness in the (a) absence and (b) presence of SRB in the soil environment.

When the steel is buried in wet soils, the transport of SRB cells, nutrients and corrosive species from the bulk environment becomes limited [Usher et al., 2014], as the soil has a blocking effect on SRB induced MIC of the steel. Thus, the corrosion rate of the steel under soil tends to decrease as the soil layer thickness increases. There is a highest corrosion rate when the soil layer thickness is 3 mm, where there is the most sessile SRB cells on the steel surface. SRB cells can reach the steel surface through gliding and twitching only in soils [Spormann, 1999]. It is more difficult for planktonic SRB cells in the bulk soil electrolyte to move to the steel surface as the soil layer
thickness increases. Thus, the number of SRB cells would decline rapidly with the increasing soil layer thickness. The decreased counts of sessile SRB cells on the steel surface result in the reduced corrosion rates. When the soil layer thicknesses exceed 10 mm, the formed surface films, including both biofilm and corrosion products, become loose and porous. This structure permits diffusion of nutrients and ions to go through the surface film towards the steel surface, maintaining a limited growth of sessile SRB cells on the steel surface under the soil layer. Once the SRB biofilm forms, the sessile SRB cells can affect the local environment, making it corrosive to steel. Therefore, although the MIC rate of the steel decreases with the increasing soil layer thickness, the corrosion rate of the steel is still much higher than that in the absence of SRB.

2.2.6. **Summary**

In the sterile soil solution, the average corrosion rates of X52 pipeline steel are 0.41 mm/y, 0.28 mm/y and 0.12 mm/y when the solution contains dissolved O$_2$ of 7.7 ppm (open to air), 0.4 ppm (purging with 5% CO$_2$) and 3.9 ppm (purging with N$_2$), respectively. The presence of SRB in the solution remarkably increases the steel corrosion. The average corrosion rates are up to 0.72 mm/y, 2.15 mm/y and 1.46 mm/y, respectively, under the three gassing conditions.

The SRB induced MIC of pipeline steel depends on the aeration of the soil solution. SRB can still survive by forming a biofilm even in the presence of O$_2$ to cause MIC, but at a low corrosion rate. In anaerobic solutions, the SRB enhanced corrosion is significant. When the soil solution contains dissolved CO$_2$, the SRB growth is further enhanced, and there is a synergism of SRB and CO$_2$ on the enhancement of the steel corrosion. This finding is the first of its kind, and is worthy of further investigation.

The SRB can grow well in wet soil, leading to MIC of the pipeline steel. Compared to steel corrosion at an average rate of 0.05 mm/y in the sterile soil environment, the corrosion rate of the steel in the presence of SRB is up to 0.28 mm/y in the soil with a mass ratio of soil to water of 5:3. SRB accelerate corrosion of the steel remarkably. The increase of the water content in soil favors the growth of SRB, making more SRB cells adhering to the steel surface and accelerating the steel MIC.

The corrosion rate of the steel is affected by the soil layer thickness. There is the largest corrosion rate when the soil layer is 3 mm in thickness in the presence of SRB. As the soil layer thickness
increases, the SRB induced MIC rate is reduced. The SRB cell counts also decrease with the increasing soil thicknesses due to the blocking effect of the soil layer on transport of SRB cells and nutrients towards the steel.

### 2.2.7. Implication

The microorganism such as SRB available in the soil plays an essential role in accelerated corrosion of pipeline steel. This enhanced corrosion by microorganism must be included in the model developed to predict long-term corrosion progression. Moreover, the structural integrity model should consider the role of microorganism in pipeline corrosion and the resulting degradation of the pipeline structure.

### 2.3. Microbiologically Induced Pitting Corrosion of the Decommissioned Pipeline

#### 2.3.1. Research background

Growth of corrosion pits in the presence of SRB is of great importance to integrity and safety of pipelines [Bartling, 2016]. The soil provides a favored environment for SRB to survive. Compared to the soil environment, the corrosive environment inside the corrosion pits would be different due to various factors such as limited transport of nutrients, deposit of corrosion products, etc. [Usher et al., 2014]. As MIC is directly related to the biofilm formation, the biofilm including extracellular polymeric substances (EPS) and corrosion products affect the growth of corrosion pits. To date, the results about the pit growth on decommissioned pipelines in SRB-containing environments have been unavailable. Moreover, the reported work on MIC was mostly conducted in aqueous solutions such as the simulated soil solutions. The pipeline corrosion occurs in wet soils much more frequently in reality than in an aqueous solution. It is realized that the corrosion of steels would experiences different processes in the two environments.

Furthermore, the perforations, once generated on the decommissioned pipeline, would further grow in the corrosive environment, especially in the presence of microorganism such as SRB. The growth mode of the initial perforations is critical to the degradation of structural integrity of the pipeline. However, none has been conducted to investigate the corrosion growth of initially generated perforations in the soil environment. This knowledge gap would be filled from the research in this topic.
The research outcomes in this topic provide mechanistic and kinetic information about pitting corrosion on decommissioned pipeline in SRB-containing wet soil. The pit growth rate along different directions on the pipeline obtained from the investigation would be used to predict the localized corrosion mode over long-term decommissioning in the soil. At the same time, the further growth mode of initial perforations generated on the pipeline is determined.

2.3.2. Research methods

The steel specimens, wet soil and the used SRB are identical to those described in section 3.2.2. The experimental setup for electrochemical corrosion measurements and the top view of the X52 steel specimen are shown in Fig. I-18, where three artificially created pits with the depths of 1 mm, 2 mm and 4 mm, respectively, are present. Generally, the pits are shaped of a cylindrical top and a conical bottom part. In reality, the geometry of bacterially induced corrosion pits is approximately semispherical, especially at the bottom part of the pits [Little and Lee, 2007]. Thus, the artificial pits machined in this work are representative of the corrosion pits in SRB-containing environments.

Fig. I-18. Schematic diagram of the experimental setup for (a) electrochemical corrosion measurements on the specimen surface and three artificially created pits, where WE, RE and CE refer to working, reference and counter electrodes, respectively. (b) Top view of the X52 steel specimen.
2.3.3. Pitting corrosion of pipeline steel in SRB-containing wet soil

Fig. I-19 shows the fluorescence images of live and dead staining of adherent SRB cells on the steel specimen and the pits with varied depths after 21 days of testing in the wet soil containing SRB, where green and red colors indicate live and dead SRB cells, respectively. It is seen that both live and dead SRB cells are extensively present on the specimen surface (Fig. I-19a), indicating that SRB survive well on the steel in wet soil. The number of the live and dead SRB cells decreases in the pit of 1 mm in depth, compared to that on the specimen surface, as shown by the color changes in Fig. I-19b. With the increased pit depth to 2 mm and 4 mm, the numbers of live and dead SRB cells in the pits further decrease (Figs. I-19c and I-19d). The results show that, although SRB can reach the pit bottom, the SRB activity decreases with the increasing pit depth.

Fig. I-19. Fluorescence images of live and dead staining of adherent SRB cells on the steel specimen surface and the pits with varied depths after 21 days of testing in the wet soil containing SRB, where green and red colors in the fluorescence images indicate live and dead SRB cells, respectively (a) specimen surface, (b) 1 mm, (c) 2 mm, (d) 4 mm.
Fig. I-20 shows the 3D views of the steel specimen, along with the depth and width changes of the pits with varied depths, upon removal of corrosion products after 21 days of testing in the presence of SRB in the soil. Corrosion of the steel specimen is obvious (Fig. I-20a). The surface topographic fluctuations increase from an average of about 15 µm to 30 µm, with some local pits detected. However, the depth changes of the pits before and after corrosion testing are marginal. There is apparent corrosion around the pit mouth, as seen in Figs. I-20c, I-20e and I-20g.

Fig. I-20. 3D views of the X52 steel specimen, along with the depth and width changes of the pits with varied depths, upon removal of corrosion products after 21 days of testing in the presence of SRB in the soil (a) specimen surface, (b, c) 1 mm deep pit, (d, e) 2 mm deep pit, (f, g) 4 mm deep pit.

Topographic analysis is conducted on the specimens and the results are shown in Fig. I-21, where the pit growth rates in the horizontal and vertical directions are derived from the 3D morphological characterizations after 21 days of testing in the absence and presence of SRB in the soil. The vertical and horizontal directions refer to the corrosion growth along the pit depth and width.
directions, respectively. It is seen that, in both soil environments, the pit growth rate in the horizontal direction is much higher than that in the vertical direction. This suggests that corrosion pits, once formed, tend to increase their width more rapidly than the depth no matter if SRB are contained. Moreover, the pit growth rate in the presence of SRB is higher than those in the absence of SRB, indicating the enhanced effect of SRB on pitting corrosion. For example, the horizontal growth rates of the pit (4 mm in initial depth) are 15.5 mm/y and 13.8 mm/y in the presence and absence of SRB, respectively. As a comparison, the vertical growth rates of the pit are 0.6 mm/y and 0.2 mm/y in the presence and absence of SRB in the soil, respectively. Generally, the pitting rate decreases as the pit depth increases. The dependence of the pit growth rate on the pit depth in the presence of SRB is consistent with the SRB cell counts as measured in Fig. I-19, where the number of bacterial cells reduces as the pit depth increases.

![Graphs showing pit growth rates](image)

**Fig. I-21.** Pit growth rates in the (a) horizontal and (b) vertical directions derived from the 3D morphological characterizations after 21 days of testing in SRB-containing soil, where Control and SRB refer to the SRB-free and SRB-containing soils, respectively.
2.3.4. Mechanism of pit growth on decommissioned pipeline steel in SRB-containing wet soil

The MIC occurring on either outside or inside the pits is closely related to the biofilm formation on the steel surface [Videla, 2000]. An important finding from this work is that, compared to the steel surface (i.e., outside the pit), there are fewer SRB cells inside the pits. As the pit depth increases, the count of sessile SRB cells reduces, as shown in the fluorescence images in Fig. I-19. During testing, there is a limited transport of SRB cells and nutrients contained in the soil towards the pits. In particular, clay soil, such as the one used in this work, makes the water diffusion and migration of SRB cells and nutrition towards the pits, especially the pit bottom, difficult [Usher et al., 2014]. SRB cells can glide or twitch to the steel surface, resulting in limited quantity of SRB cells to reach the pit bottom. Moreover, it is relatively difficult for the nutrition such as organic carbon, sulfate and other salts to transport into the pits. The reduced number of SRB cells with the increasing pit depth serves as a direct evidence that the bacterial activity drops due to the limited transport of nutrients to the pit bottom. The SRB presenting extensively on the specimen surface actively participate in MIC under the biofilm. However, the MIC is reduced inside the pits due to limited quantity of SRB cells, as confirmed by the 3D morphological observation of the pit growth on the steel.

Furthermore, compared to the vertical growth (depth) of the pits, the pit growth along the horizontal direction (width) is much faster. This is associated with the accumulation of SRB around the pit mouth, rather than inside the pits, due to availability of nutrients and energy sources on the specimen surface.

A mechanistic model is proposed to explain the pit growth and MIC around the pit mouth on X52 pipeline steel in SRB-containing soil, as shown in Fig. I-22. While substantial SRB cells accumulate on the steel surface, there is a limited quantity of sessile SRB cells inside the pits. The growth rate of the pits in the vertical direction is much smaller than the corrosion rate outside the pit due to a lower quantity of SRB cells in the pits. At the same time, a galvanic coupling effect is generated between the inside and outside the pits, producing a separated anode (i.e., outside pits) and cathode (i.e., inside pits). The SRB cells accumulating on the steel surface enhances corrosion at the anode, accelerating the growth of the pits horizontally. At the same time, the transport of sulfate to the pits is difficult due to the “blocking effect” [Usher et al., 2014], and SRB cells in the soil can obtain sulfate easier than the pits. The electrons produced from the anode, i.e., pit mouth,
flow to the cathode, i.e., inside the pits, accelerating corrosion of the pits in the horizontal direction. The SRB cells in the pits can obtain electrons from the anode for reduction of sulfate. The galvanic corrosion induced by SRB is attributed to the heterogeneous distribution of sulfate and different activities of SRB on the steel surface compared to those inside the pits.

![Diagram of SRB corrosion](image)

Fig. I-22. A conceptual model illustrating the pit growth and corrosion around the pit mouth on pipeline steel in the presence of SRB in the soil.

2.3.5. Further growth of initial perforations on pipeline in the soil

![Diagram of test rig](image)

Fig. I-23. Schematic diagram of the test rig to investigate the corrosion growth, both internally and externally, of initial perforations on X52 pipeline steel in the absence and presence of SRB in the soil.
The test rig, as shown in Fig. I-23, was home-designed to investigate the corrosion growth, both internally and externally, of initial perforations on X52 pipeline steel in the absence and presence of SRB in the soil. The artificially created holes with the diameters of 0.1 mm, 1.0 mm, 1.5 mm, 2.0 mm and 2.6 mm, respectively, were prepared on an X52 steel plate. Array electrodes with a diameter of 1 mm were used to study the internal corrosion growth of the holes. A Zeta 3D stereoscopic microscope was used to characterize the topographic profile of the corroded steel holes, and the corrosion rates were derived during the testing time period.

Fig. I-24. Corrosion rates of the artificial holes at the mouth area along the horizontal direction and the localized corrosion rates in the vertical direction after 30 days of testing in the absence and presence of SRB in the soil, where “control” means the sterile soil.
The corrosion rates of the artificial holes at the mouth area along the horizontal direction and the localized corrosion rates in the vertical direction after 30 days of testing in the absence and presence of SRB in the soil are shown in Fig. I-24, where “control” means the sterile soil. It is seen that the corrosion rates of the steel in the horizontal direction are generally high, exceeding 19 mm/y, where the presence of SRB accelerates the corrosion. Localized corrosion in the vertical direction occurs during the horizontal corrosion of the artificial holes in the soil. However, the localized corrosion rates are much lower than the horizontal ones.

2.3.6. Summary

For X52 pipeline steel in SRB-containing wet soil, the bacterial cells tend to accumulate on the steel surface, rather than inside corrosion pits. The counts of SRB cells decrease with the increase of the pit depth due to the limited transport of SRB cells and nutrients contained in the soil towards the pits. As the pit depth increases, the corrosion rate decreases. This is attributed to the reduced number of SRB cells in the pits. Compared to the vertical growth of the pits along the depth direction, the pit growth along the horizontal direction (i.e., the width direction) is much faster. For example, the horizontal and vertical growth rates of the pit (4 mm in initial depth) are 15.5 mm/y and 0.6 mm/y in the SRB-containing soils, respectively.

For initial perforations on decommissioned pipeline in the soil, the corrosion growth along the horizontal direction at the perforation mouth area is much faster than that of the vertical direction. Thus, the perforation mouth tends to become wide due to corrosion with time. The presence of SRB in the soil accelerates the corrosion of the steel at both directions at the mouth of the initial perforations.

2.3.7. Implication

While pitting corrosion usually occurs on pipelines in microorganism-containing soils, the dominant corrosion mode would be the widening of the pit mouth, rather than deepening of the pits. At the same time, for initial perforations present on the pipeline, the further growth due to corrosion is also primarily controlled by the horizontal widening, rather than vertical deepening, at the mouth area. These conclusions are important in terms of the modeling and prediction of structural integrity of corroded pipeline during decommissioning in the soil.
2.4. Modeling of Pipeline Corrosion During Long-Term Decommissioning in Place

2.4.1. Research background

Corrosion is the primary mechanism causing reduction of the pipe wall thickness and degradation of the structural integrity of pipelines [Cheng, 2013]. It is thus critical to accurately predict the corrosion rate of the pipeline during long-term decommissioning in the soil, providing key data to model and predict the structural integrity and the potential risk associated with the pipeline decommissioning.

Pipeline corrosion in soils is very complicated, and the complexity arises from multiple affecting factors and many interacting processes that occur simultaneously. Experimental testing provides a main methodology to investigate pipeline corrosion. However, the approach suffers from the limited time period of testing and incapability of reproducing and controlling the realistic environmental conditions experiences on decommissioned pipeline in the soil. Therefore, a well-accepted method for prediction of long-term corrosion progression is to develop a mechanistic model, which is combined with numerical computation, to enable accurate determination of the long-term corrosion rate of the pipelines.

Models for soil corrosion of metals such as steel pipelines have been available in various sources. However, most of the existing models are empirical or semi-empirical [Alamilla et al., 2009], which lack a theoretical background and have a poor extrapolation performance. To date, a model applicable for prediction of long-term corrosion of decommissioned pipelines in soils, while considering the mechanistic effects of the affecting factors such as soil chemistry, temperature, dissolved oxygen, coatings, microorganism, etc., has been not available.

The research outcome of this topic develops a mechanistic model, which, by integrating mass-transfer and charge-transfer during corrosion processes, enables accurate prediction of long-term corrosion rate of pipeline steel in the soil under conditions that are relevant to pipeline decommissioning. The modeling results are verified by experimental testing and literature data.

2.4.2. Model development

The physical block of the developed corrosion model is schematically shown in Fig. I-25. The bare pipeline steel is immersed into the simulated soil solution. When oxygen is available, \( O_2 \) is
dissolved in the solution and transports towards the steel surface. In the near-neutral, anaerobic solution, the cathodic reaction is the reduction of water molecules. Dissolved O$_2$ participates in the electrochemical cathodic reaction and consumes on the steel surface. Fe$^{2+}$ is produced due to the anodic reaction and transports into the solution. H$^+$ and OH$^-$ ions in the solution maintain at an equilibrium state. When corrosion products are formed on the steel surface, both the mass transfer of corrosive species and the charge-transfer reactions are affected.

![Diagram of the physical block of the developed model](image)

Fig. I-25. Schematic diagram of the physical block of the developed model.

Assumptions are made to facilitate numerical calculations, including:

- The effect of inert chemical species on corrosion reactions is negligible. However, these species affect the solution conductivity and the electro-migration happening in the solution.
- Fe(OH)$_2$ is the primary corrosion product during corrosion of pipeline steel in neutral soil solutions. The conversion of Fe(OH)$_2$ to other chemicals is not considered [Li et al., 2006].
- One-dimensional computational domain from the steel surface to the liquid/gas interface is used for modelling.

**Electrochemical reactions and the reaction kinetics.** The current densities of the iron oxidation and the water reduction are written as the Tafel equations with a porosity, $\varepsilon$, of the corrosion products:

$$
    i_{Fe} = \varepsilon i_{0,Fe} \times 10 \left( \frac{E - E_{rev,Fe}}{b_{Fe}} \right)
$$

(I-1)
\[ i_{H_2O} = \varepsilon i_{0,H_2O} \times 10^{-\frac{E + E_{rev, H_2O}}{b_{H_2O}}} \]  

(I-2)

The oxygen reduction reaction is under mixed control, and the current density is calculated as [Gadala et al., 2016]:

\[ i_{O_2} = \alpha_{0,O_2} \frac{C_{O_2}^{ss}}{C_{O_2}^{ref}} \times 10^{-\frac{E + E_{rev, O_2}}{b_{O_2}}} \]  

(I-3)

where \( C_{O_2}^{ss} \) and \( C_{O_2}^{ref} \) are the oxygen concentrations (mol/m\(^3\)) on the steel surface and at the air/solution interface, respectively.

The reversible potentials for anodic (Fe oxidation) and cathodic (reduction of dissolved oxygen or water) reactions are expressed as:

\[ E_{rev,Fe} = -0.44 + \frac{2.303RT}{2F} \log([Fe^{2+}]) \]  

(I-4)

\[ E_{rev,O_2} = 1.229 - \frac{2.303RT}{F} \text{pH} + \frac{2.303RT}{4F} \log([O_2]) \]  

(I-5)

\[ E_{rev,H_2O} = -\frac{2.303RT}{F} \text{pH} \]  

(I-6)

where \([Fe^{2+}]\) is the activity of Fe\(^{2+}\) ions, and is set as 10\(^{-6}\) M [Gadala et al., 2016], and \([O_2]\) is the activity of dissolved oxygen on the steel surface.

The corrosion potential, \(E\), can be calculated from the charge balance on the steel surface by:

\[ \sum_{i} n_{i} i_a = \sum_{i} n_{i} i_c \]  

(I-7)

**Governing equations.** The distribution and transportation of corrosive species in the solution follow the mass conservation and electro-neutrality equation, respectively [Dickinson et al., 2011]:

\[ \frac{\partial (\varepsilon C_j)}{\partial t} + \frac{\partial (N_j)}{\partial x} = \varepsilon R_j \]  

(I-8)
\[
\sum_{j=1}^{n} C_j z_j = 0
\]  
(I-9)

where \( C_j \) is the concentration of species \( j \), \( N_j \) is the flux of species \( j \), \( R_j \) is the source or sink of species \( j \) due to chemical reactions, \( t \) is time, \( x \) is spatial coordinate, and \( \varepsilon \) is volume porosity of the scale. The flux of the species is given by the Nernst-Planck equation, but the electro-migration and convection components are negligible:

\[
N_j = -k D_j \frac{\partial C_j}{\partial x}
\]  
(I-10)

The overall conservation equation is rearranged as:

\[
\frac{\partial (\varepsilon C_j)}{\partial t} - D_j \frac{\partial^{2} (\varepsilon^{1.5} C_j)}{\partial x^{2}} = \varepsilon R_j
\]  
(I-11)

**Initial and boundary conditions.** The initial condition considers the moment that the steel is yet immersed in the soil solution, and corrosion does not happen. The solution is saturated with dissolved oxygen. The concentrations of \( \text{H}^+ \) and \( \text{OH}^- \) ions are uniform in the solution, and their concentrations can be calculated with the input of the measured solution pH.

Two boundaries are considered in the model. One is the air/solution interface, and the other one is the steel/solution interface. At the air/solution interface, the \( \text{O}_2 \) concentration is constant at a saturated condition. At the steel/solution interface, the concentrations of \( \text{Fe}^{2+} \), dissolved \( \text{O}_2 \) and \( \text{OH}^- \) are determined by interfacial charge-transfer reactions.

**Numerical solution.** The finite difference method is used to solve the transient ordinary and partial differential equations, with non-uniform space interval grids created for the one-dimensional geometry and the central implicit scheme for transient calculation.

**2.4.3. Modelling results**

The corrosion rate of X52 pipeline steel in the aerobic simulated soil solution is determined by the developed model, and the result is shown in Fig. I-26, where the measured results are also included for comparison. It is seen that the modeled corrosion rates of the steel fit well with the experimental results. The steady-state corrosion rate of the steel in the soil solution is about 0.1 mm/y.
Fig. I-26. Comparison of the corrosion rate of pipeline steel in the simulated soil solution determined by the developed model with that measured in experimental testing.

Fig. I-27. Modeling and experimentally measured corrosion rates of the steel as a function of the dissolved oxygen concentration in the soil solution.
For soil corrosion of steels, the dissolved oxygen concentration is critical to the corrosion mechanism and growth kinetics. The corrosion kinetic parameters for two dissolved oxygen concentrations, i.e., 3.9 ppm and 0.4 ppm, are determined using the developed computational program. When the dissolved oxygen concentration is 3.9 ppm, the corrosion of the steel involves the reduction of dissolved oxygen. However, when the dissolved oxygen concentration is 0.4 ppm, the reduction of water becomes the dominant cathodic reduction. The calculated and experimentally measured corrosion rates at various dissolved oxygen concentrations are shown in Fig. I-27. It is seen that, although a deviation exists between the experimental and modeling results, especially at high dissolved oxygen concentrations, the model can predict the trend with a reasonable range that the presence of dissolved oxygen increases the corrosion rate of the steel.

The prediction of pipeline steel corrosion in soils is also conducted by comparison with existing models and field-collected corrosion data. Fig. I-28 shows the corrosion rate and thickness loss of the steel in the soil solution from the developed model, as compared with the prediction results from DNV model [Veritas, 2015] \( p = 0.2\times t^{1/2} \), where \( p \) is the maximum corrosion depth, and \( t \) is time) and the field data from National Bureau of Standards (NBS) database [Romanoff, 1957]. For long-term corrosion prediction, two scenarios are considered, i.e., (1) the corrosion products are compact and protect the steel from further corrosion, and (2) there exists a minimum porosity of the corrosion product scale at which the corrosion rate keeps constant at a low value during the long-term exposure, and the minimum scale porosity was set as 0.3. It is that. When a minimum porosity of the corrosion product scale is considered, the developed model gives an accurate prediction of the corrosion rate of the steel, which is well consistent with the field data. Instead, the existing DNV model tends to give larger corrosion rates than the actual values collected from the field. Moreover, the comparison shows the importance of the corrosion product scale in corrosion of the steel. For long-term corrosion of decommissioned pipeline in the soil, the formation of corrosion scale provides somewhat protection against the further corrosion. It is seen from the model result that the corrosion rate is low over the long-term exposure in soils that uniform corrosion by itself does not constitute a vital threat to the integrity of the decommissioned pipeline in the soil.
Fig. I-28. Corrosion rate (a) and thickness loss (b) of the steel in the soil solution from the developed model, as compared with the prediction results from DNV model and the field data from National Bureau of Standards database.

As demonstrated above, when the soil contains microorganism such as SRB, the steel corrosion would be accelerated at an appreciable level. Thus, the developed corrosion model must consider
the enhanced effect of microorganism on corrosion of the pipeline steel. Or else, the modeling prediction does not give accurate results in terms of the corrosion progression of decommissioned pipeline in the presence of microorganism in the soil. Therefore, an accurate prediction of actual corrosion rate of decommissioned pipeline in SRB-containing soil is based on determination of the SRB effect factor.

By dividing the corrosion rate of X52 pipeline steel in SRB-containing soil by that in the sterile soil, the SRB effect factor, i.e., $F_{SRB}$, is determined. This work also confirms that $F_{SRB}$ is a function of the soil layer thickness, as seen in Fig. I-17. Fig. I-29 shows the determined $F_{SRB}$ as a function of the soil layer thickness, where an exponent relationship is derived as:

$$F_{SRB} = 9.8 + 83.1 \times 0.6^{\text{thickness}}$$  \hspace{1cm} (I-12)

**Fig. I-29. The determined $F_{SRB}$ as a function of the soil layer thickness.**

Fig. I-30 shows the modeling results of corrosion rate and thickness loss of X52 pipeline steel during long-term decommissioning in the SRB-containing soil. Compared to the sterile soil in Fig. I-28, the corrosion rate of the steel increases remarkably in the presence of SRB. Moreover, as the soil layer thickness increases, the corrosion rate of the steel decreases, with the exception that the steel corrosion reaches a maximum rate at the soil layer thickness of 3 mm. As the soil layer thickens, the corrosion rate of the steel tends to close each other. For example, when SRB are present in the soil, the pipeline wall thickness could lose 2.5 mm after 10 years of burial in the soil exceeding 25 mm in thickness.
Fig. I-30. The predicted (a) corrosion rate and (b) wall thickness loss by the developed model with consideration of the SRB effect on enhanced corrosion as a function of time.

2.4.4. Summary

A mechanistic model, which combines both mass transfer of corrosive species in the soil medium and electrochemical charge-transfer reactions on the steel surface, is developed, which enables prediction of the corrosion rate of X52 steel pipeline during long-term decommissioning in the soil. The modeling results are validated by the corrosion rates obtained from experimental tests and the data collected from available literature. The model considers the essential of the porosity of the
corrosion product film generated on the steel and the accelerating corrosion in the presence of SRB in the soil. The effects are quantified, and the model is modified based on experimentally determined data. The long-term corrosion rate and the loss of pipeline wall thickness as a function of time are predicted.

2.4.5. Implication

The corrosion growth of decommissioned pipeline in the soil is one of the primary mechanisms resulting in structural degradation of the pipeline. Thus, development of a mechanistic model for accurate prediction of the long-term corrosion progression of the pipeline is key to evaluation of the pipeline structural integrity. The developed model in this work provides the corrosion rates of X52 pipeline steel in the soil and simulated soil solution as a function of time with considerations of the mass transfer and charger transfer steps occurring during steel corrosion, the porosity of the corrosion product layer, and the accelerating effect of microorganism (i.e., SRB) on the steel corrosion. At the same time, the roles of dissolved oxygen concentration, soil layer thickness, the moisture content in the soil, etc., are investigated and can be modeled quantitatively.

As stated, soil corrosion of pipelines is a complex phenomenon. In addition to the affecting factors mentioned above, there are more factors also play a role in corrosion of the steel. These include, but not limited to, multiple bacterial communities, temperature, salinity, minerals, etc. Moreover, these factors could interact each other and cause more complicated processes. Therefore, the model development always has a space to improve. At the same time, it is realized that the corrosion-affecting factors can be ranked in terms of their importance to corrosion mechanism and rate, especially the long-term corrosion progression. In this work, emphasis has been placed on the essential role of SRB in accelerated corrosion of the pipeline steel. Both experimental testing and literature review demonstrate that the strategy in factor-selection is reasonable, and the modeling results are reliable.

The modeling results of the long-term corrosion rate of the pipeline steel serve as the foundation to evaluate and predict the structural integrity of the decommissioned pipeline, which is covered in the theme of Structural Integrity.
3. Structural Integrity

3.1. Introduction

Steel pipelines are being used by the energy-related industry in North America as the primary means of transporting crude oil, natural gas and petro-chemical products. In Canada alone, the energy pipelines extend along more than 700,000 km [Nazemi and Das, 2010]. The majority of these pipelines run underground. At the end of a pipeline’s useful life, it is decommissioned and may be removed or left in place underground. When a decommissioned pipeline is left underground, various corrosion protection methods are terminated, no longer maintained, or degrade over time. As a result, the pipeline will begin to corrode due to environmental and in-situ conditions. Corrosion is the primary cause of structural degradation of decommissioned pipelines. Such degradation results in a reduction in the material strength and stiffness of the pipe section, and the pipeline may no longer be capable of bearing the imposed surface loading; in particular, heavy traffic, construction equipment, and farming machinery. Potential collapse of buried pipelines poses a major risk to both the public and the environment.

Extensive research has been carried out over the last several years to study the behavior of pipelines under different types of loading. Ozkan and Mohareb [2009] tested pipe sections under bending, tension, and internal pressure, and verified the use of finite element analysis for predicting the pipe structural capacity and buckling behavior. A few investigations have been conducted on different forms of pipeline damages such as wrinkling, denting, and fracture under operational loads and low-cycle fatigue [Das, Cheng and Murray, 2007]. The effects of significant damages due to local buckling or construction errors were assessed without accounting for the long-term effects of wall thinning due to global, local, or pitting corrosion. Behavior of corroded in-service pipelines has also been investigated, with operational internal pressure being the primary load acting on the pipeline [Dewanbabee, 2009]. The buckling behavior of cylindrical shells under pure bending and external pressure has been numerically investigated [Ghazijahani and Showkati, 2013] without accounting for soil-pipe interaction effects. Finite element investigations on buried concrete pipes subjected to surface live loading indicate that soil-pipe interaction is necessary to study pipelines at shallow burial depths [Noor and Dhar, 2003]. Various studies have been conducted on pipelines in operation subjected to both static and moving surface live loading [Kabir, 2006; Neya et al., 2017].
These studies indicate that pipe stresses are highly dependent on both the soil-pipe relative stiffness and the pipe burial depth, with maximum stresses at the pipe crown. However, the presence of internal pressure reduces the net pressure differential. Full-scale experimental testing has been conducted on flexible empty pipes subjected to live loads [Arockiasamy, Chaallal and Limpeteeprakarn, 2006]. The tests were limited to intact pipe sections and did not account for long-term strength and stiffness reduction due to corrosion. In spite of the many studies reported in the literature, there exist very limited available information on the behavior and degradation of the structural integrity of decommissioned steel pipelines left in place underground.

3.2. Problem Formulation

The initial step in determining the long-term structural integrity of decommissioned pipelines is assessing and understanding the static response of the pipeline immediately after decommissioning. The static response of a pipeline subjected to surface live load is dependent on a variety of factors including load geometry and location, burial depth, soil-to-pipe relative stiffness, pipe section material and mechanical properties, and soil-pipe interaction. Imposed loads are transferred from the soil to the pipe section through a process known as soil arching. Soil arching describes the state where an unstable soil mass (due to translation or yielding) transfers stress to an adjacent rigid body [Terzaghi, 1943]. In a soil block, this rigid body may consist of adjacent stable soil or an adjacent pipeline with stiffness much greater than that of the soil.

As the burial depth is reduced, the path from the surface load to the pipe section becomes shorter and more direct. Since the pipe section is much stiffer than the surrounding soil, the load attracted by the pipe section is expected to increase. Based on this fundamental observation, the initial analysis consists of investigating the effects of depth of soil cover on the resulting stresses and deformations of the pipe section. A reasonable critical depth is then selected to investigate the effects of wall thickness and soil stiffness (Fig. II-1). Investigating the effects of the wall thickness serves two purposes: 1) determining the initial response of pipe sections of different wall thicknesses, and 2) investigating the long-term effects of global corrosion. Global corrosion involves the uniform wall thinning of a pipe section. Within the two analyses conducted, four separate pipe sections are used, enabling the effect of pipe diameter-to-thickness ratio to be observed simultaneously. Pipe diameters of 20”, 24”, 30”, and 34”, with standard (Sch. 40s) wall thicknesses of 3/8” correlating with the nominal pipe size, were used for all analyses.
Fig. II-1. Analyses to be considered: effects of burial depth, wall thickness, and soil stiffness.

3.3. The Finite Element Model

All models developed for the subsequent studies were analyzed using the commercial finite element software package ABAQUS [Dassault Systèmes, 2014]. ABAQUS is commonly used throughout the industry for finite element analysis of buried pipelines subjected to a variety of loadings including surface loading, soil movement, and internal pressure, among others [Kabir, 2006; Dewanbabee, 2009]. Results from these previously conducted analyses have been validated and corroborated with their respective experimental data. ABAQUS also has the capability of modelling and analyzing material and geometric nonlinearities.

3.3.1. General model and mesh properties

Element selection. The pipe section was modelled using the general purpose, four-node, first-order S4 shell elements with six degrees of freedom per node. These elements are recommended for thin-shell applications and have an inherent assumed large-strain formulation, allowing for use in analyses involving in-plane bending [Sadowski, 2013]. Thin-shelled structures are defined in ABAQUS as having a thickness ratio greater than 1/15 compared to a characteristic length [Dassault Systèmes, 2014]. The characteristic length of interest for the pipe sections is considered to be the pipe radius. For a 20” pipe section, a standard 3/8” wall thickness equates to a radius-to-thickness ratio of 26.7; much higher than the limiting value.

The soil block was modelled using second order, ten-node, tetrahedron C3D10 solid continuum elements. The main influencing factor for the soil elements was the ability to obtain a high-quality mesh. Due to the transition from rectangular geometry of the loading and soil block to the circular geometry of the pipe section, tetrahedron elements proved most advantageous. Tetrahedral
elements allow for ease of meshing around circular boundaries and are less sensitive to initial element distortion [Dassault Systèmes, 2014].

**Model size and boundary conditions.** The final model geometry and size were chosen such that the influence due to proximity of the boundary conditions is negligible. As a result, the model is representative of a pipeline buried within an infinite soil medium. The four side surfaces of the soil block were modelled using roller boundary supports, preventing translation normal to the surface while allowing for vertical displacement. The base of the block was hinge-supported, preventing translation in all orthogonal directions. The boundary conditions are applied to the surface of the model, as opposed to individual nodes, implicitly restricting rotations.

The model geometric size was then optimized to negate the constraining effects of the proximity of the boundary conditions. An initial soil block size L x W x H equal to 5 m x 20 m x 5 m was modelled. The width of 20 m is equal to the length of Alberta’s CL-800 truck plus an additional 1 m edge distance on both ends. The length, width and height of the soil block were then individually increased, and the resulting stresses were plotted. An optimal model size was determined when the maximum stresses converged. Based on the convergence analysis, the optimal model size was selected as L x W x H equal to 20 m x 28 m x 12 m (Fig. II-2).

![Fig. II-2. General model layout and geometry.](image)

**Mesh refinement.** The accuracy of the analysis is directly related to the finite element mesh quality and refinement. In general, as the mesh is refined (size of elements reduced/number of degrees of freedom increased), the results tend to converge towards the true solution. Two mesh
refinements were performed separately: one for the pipe section and one for the soil block. For mesh refinement of the pipe, a 34” pipe beam section was analysed. The pipe was modelled with fixed-fixed end conditions and subjected to a uniformly distributed load. The resulting stresses were compared to theoretical values obtained using Timoshenko beam theory. The aspect ratio of the elements was maintained at 1:1 and the number of elements around the circumference of the pipe section was increased until convergence of the pipe stresses was achieved. Convergence occurred at approximately 30 elements around the pipe circumference.

Mesh refinement of the soil block was performed assuming a biased mesh, allowing for increased local mesh refinement around the location of the pipe section and near the points of load application (Fig. II-3).

![Finite element mesh](image)

(a) Sectional elevation.

(b) Plan view

**Fig. II-3. Finite element mesh.**
Since the influence of the boundary conditions is negligible, the boundaries of the block can afford a coarser mesh. A biased mesh greatly reduces the number of elements used and the computational time required. The degree of bias was then increased, increasing the number and reducing the size of the elements at the critical locations. Convergence of the stresses occurred at approximately 230,000 elements. The final model geometry and mesh refinement was completed for the largest pipe section, the 34” diameter pipe. The same geometry and mesh size (number of elements around the pipe section and the degree of bias) were used for all subsequent models with smaller pipe diameter. This is a conservative assumption resulting in smaller element size and greater number of degrees of freedom for smaller diameters.

3.3.2. Material properties

Pipe section. Pipelines can be composed of various materials depending on the contents to be transported, soil conditions and stress requirements. For the purpose of this investigation, the same API 5L X52 pipeline steel used in Enbridge Line L3 was selected. This steel specification is commonly used for transporting water, oil, gas, and chemicals throughout the petroleum industry. The pipeline steel was modelled as a homogenous elastic-plastic material with a yield strength, $\sigma_y$, of 360 MPa, modulus of elasticity, $E$, of 207,000 MPa, and Poisson’s ratio, $\nu$, of 0.3 [CSA Z662-15, 2015; CSA Z245.1-18, 2018]. Material plasticity beyond the yield stress was modelled assuming a Ramberg-Osgood material relation between stress, $\sigma$, and strain, $\varepsilon$ given by:

$$\varepsilon = \frac{\sigma}{E} + \alpha \left(\frac{\sigma}{\sigma_y}\right)^n$$

(II-1)

where the parameter $\alpha$ represents the yield offset strain, assumed to be 0.2% for ductile steel; and $n$ was modified such that the resulting stress strain curve fits with literature curves for equivalent steel grade API 5L X52 [Trifonov, 2015]. The nominal stresses and strains were then converted to true stress and strain to be used in ABAQUS.

Soil. It is common for multiple soil layers to be present in the overburden above a buried pipeline. However, for the burial depths analyzed (< 3 m), it is reasonable to assume the overburden consists of a single homogenous soil layer. Thus, the models were developed assuming a single homogenous soil block with uniform material properties. The soil material was modelled using an
elastic-perfectly plastic Mohr-Coulomb plasticity model. This model assumes that failure is controlled by the maximum shear stress (Fig. II-4).

![Mohr-Coulomb material model](image)

**Fig. II-4. Mohr-Coulomb material model**

The Mohr-Coulomb model has been used extensively in finite element modeling of soil-pipe interaction analysis and is recommended by ABAQUS for geotechnical problems subjected to monotonic loading [Dassault Systèmes, 2014]. The soil material was assumed to be soft clay. Soft clay is present in abundance in North America and serves as a critical soil material with its low modulus of elasticity and internal friction angle compared to other types of soil. Mohr-Coulomb parameters used are summarized in Table 1 [Lee, 2010; Subramanian, 2010].

**Table II-1. Soil properties.**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value Used in Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mass Density, $\gamma$ (kg/m³)</td>
<td>18</td>
</tr>
<tr>
<td>Young’s Modulus, $E$ (MPa)</td>
<td>25</td>
</tr>
<tr>
<td>Poisson’s Ratio, $\nu$</td>
<td>0.3</td>
</tr>
<tr>
<td>Friction Angle, $\phi$ (°)</td>
<td>25</td>
</tr>
<tr>
<td>Dilation Angle, $\psi$ (°)</td>
<td>2</td>
</tr>
<tr>
<td>Cohesive Strength, $c$ (kPa)</td>
<td>100</td>
</tr>
</tbody>
</table>
3.3.3. Soil-pipe contact definition

The stress state of a pipeline subjected to surface loading is highly dependent on the applied load path. A significant factor influencing the load path is the soil-pipe interaction properties. Due to thermal expansion and contraction, soil shrinkage, pipeline corrosion, and formation of drainage channels, it is unrealistic to assume that a perfect bond exists between the pipe and soil. Therefore, it is more realistic to employ a frictional slippage model between the pipe surface and the surrounding soil. This contact definition will allow for finite soil slippage to occur should the frictional resistance be overcome.

Frictional soil-pipe interaction was modelled in ABAQUS using a surface-to-surface contact definition and employing the penalty enforcement method. Penalty enforcement allows for various contact slip formulations including friction, shear stress and elastic slip. Slip between the soil and pipe surface was defined using a coefficient of static friction. The coefficient of friction, $\mu$, was determined based on the ASCE ALA Guidelines for Design of Buried Steel Pipe [American Lifelines Alliance, 2001] and is given as:

$$\mu = \tan(f\varphi)$$

(II-2)

The coating dependent factor, $f$, is attributed to the surface condition of the pipe section. A factor of 0.8 was used to represent a rough surface due to corrosion. Therefore, the coefficient of friction used was 0.36. Frictional contact was applied in the longitudinal and circumferential directions of the pipe section, while ‘hard’ contact was applied in the normal direction (Fig. II-5). Hard contact ensures zero penetration of the two surfaces while allowing for separation as the system deforms.

Fig. II-5. Soil-pipe contact definition
3.3.4. Types of Loading

The pipeline is subjected to two types of loading: 1) dead load due to self-weight of the pipeline and the soil overburden, and 2) surface live load due to traffic. The self-weight of the pipeline and soil overburden was applied as a gravity load to the entire model. Material densities were defined. The surface live load was taken as axle loads due to either an Alberta CL-800 truck (Fig. II-6) or a heavy farming or construction equipment (Fig. II-7). The axle load was applied as pressure over the contact area of the two wheels at each axle location, with a dynamic load allowance factor (DLA) appropriate for arch-type buried structures as per the Canadian Highway Bridge Design Code (CHBDC) [CSA-S6-14, 2014]:

\[ DLA = 0.4(1 - 0.5D_E) \]  \hspace{1cm} (II-3)

where \( D_E \) is the burial depth. The dynamic load allowance factor is not taken as less than 0.1, which corresponds to a burial depth of 1.5 m. Therefore, at burial depth shallower than 1.5 m, the DLA will increase with decreasing depths [CSA-S6-14, 2014]. The loads are applied considering the truck travelling perpendicular or parallel to the axis of the pipe section. For a truck travelling perpendicular to the pipe section, the maximum axle load is applied directly over the axis of the pipe section (Fig. II-8).

Fig. II-6. Alberta CL-800 truck loading diagram.
3.4. Parametric Study Results and Discussion

Using the afore-mentioned modelling techniques, geometries, material properties, and loading conditions, an optimal baseline soil-pipe finite element model was developed and used in parametric studies investigating the factors influencing the response of decommissioned pipelines to surface traffic loads. The effects of the following parameters were investigated: 1) the burial depth, 2) the pipe diameter, 3) wall thickness and diameter-to-thickness ratio, 4) soil stiffness, 5) surface live load types and conditions. It is recognized that the soil-pipe system has the potential to experience large local deformations as the burial depth and wall thickness are reduced. Therefore, non-linear geometric finite element analyses were performed to obtain the resulting stresses and deformations.

The primary results of interest include the pipe section ovalization and the resulting normal stresses: longitudinal stresses along the pipe axis arising from longitudinal bending, and circumferential hoop stresses arising from the external pressure differential and transverse soil friction. The results
extracted are located directly beneath the point of one wheel load application. Stresses were obtained at the extreme fibre at the pipe crown, invert, and springline (i.e., the top-most, the bottom-most, and the mid-height points of the pipe cross-section, respectively). The maximum stress for all pipe sections analyzed was found at the pipe crown. Therefore, for the sake of brevity, only stresses at the pipe crown are plotted and discussed below. Pipe section ovalization is defined using the Modified Iowa Equation as the vertical change in pipe diameter divided by the pipe original (undeformed) outer diameter [American Lifelines Alliance, 2001].

3.4.1. Effects of burial depth

The burial depth is varied between 3.0 m and 0.25 m in 0.5 m increments. Wall thickness is maintained at the standard 3/8” for all diameters. As can be seen in Fig. II-9, both the longitudinal and hoop stresses increase in magnitude with the decrease in burial depth. The increase in stresses is non-linear, rather naturally logarithmic, indicating that the burial depth is a significant factor affecting the resulting stress state.

As the burial depth decreases, the surface live load path to the pipe becomes shorter and more direct. As such, the pipe attracts more load and the resulting stresses increase. The maximum stress is compressive and is located at the pipe crown. The compressive stresses are due to both global and local bending of the section. At burial depths shallower than 1.0 m, the resulting stresses begin to increase rapidly and become more localized under the wheel loads (Figs. II-9 and II-10). However, it is noted that the maximum stresses remain below the pipe yield stress. The 1.0 m threshold is interpreted visually from the graphs. At burial depths less than 1.5 m, the dynamic load allowance factor increases with decreasing burial depth, amplifying the resulting stresses and strains. Therefore, 1.0 m can be considered a step change in the dynamic load allowance factor.

Further, pipes of smaller diameters generally experience larger stresses at deeper burial depths. As the pipe diameter is reduced, the moment of inertia of the pipe section also reduces, increasing the global bending stresses. However, as the burial depth continues to decrease, smaller pipe diameters experience lower stresses. This transition from smaller to larger diameter pipes governing the stress state occurs at an approximate depth of 0.5 m. At shallow burial depths, the surface load acts similar to a concentrated force applied directly to the pipe section. As a result, local bending stresses begin to govern the stress state. By maintaining a constant wall thickness for all pipe
diameters, smaller pipe diameters have a larger local wall stiffness. The increases in stresses due to reduction in pipe diameter are offset by the simultaneous reduction in diameter-to-thickness ratio. This observation is further confirmed in the wall thickness study presented in Subsection 3.4.2.

Fig. II-11 illustrates the effect of burial depth on the pipe ovalization. As the burial depth decreases, vertical ovalization also increases. As the pipe diameter decreases, smaller pipes experience smaller ovalization while the increasing trend is near identical. Smaller pipe sections have inherently smaller absolute dimensions and an increased local wall stiffness, resulting in smaller ovalization.

![Graph showing effect of burial depth on stresses](image)

**Fig. II-9. Effect of burial depth on stresses at the crown of pipes of different diameters.**

![Graph showing ovalization](image)

**Fig. II-10. Longitudinal stresses in a 34” diameter pipe for deep and shallow overburden.**
3.4.2. Effects of pipe wall thickness and diameter-to-thickness ratio

Based on the burial depth analysis, the stresses begin to become significant at approximately 1.0 m of soil cover. Therefore, the effect of wall thickness is investigated using 1.0 m soil cover. The wall thickness is varied between 5/8” (schedule 30 for 34” and 30” pipe section; kept consistent for 24” and 20” pipe section) and 1/16” at the extreme ends. Typical pipe sections have a common minimum thickness of approximately 1/4”. Therefore, thicknesses less than 1/4” can be considered a result of uniform global corrosion.

Fig. II-12 shows that as the wall thickness is reduced, both the longitudinal and hoop stresses increase. However, the hoop stresses increase to a maximum value and then begin to decrease, while the longitudinal stresses continue to increase in magnitude. The increase in longitudinal stresses is due to the reduction in the section’s moment of inertia, increasing the global and local bending stresses. The reduction in the hoop stresses is due to the localization of the load and the reduction in the section’s overall stiffness.
Fig. II-12. Effect of wall thickness on stresses at the crown of pipes of different diameters.

As the wall thickness is reduced, the load effects become more localized at the location of the wheel load. With a thick pipe wall, the pipe stiffness can be great enough to cause increase in the length of pipe affected by the maximum stresses resulting from the surface loads (Fig. II-13a). As the stiffness of the pipe decreases, the local deformations of the section increase and the surface loads act independently of one another (Fig. II-13b). Further, as the wall thickness is reduced, the stiffness of the pipe section relative to the surrounding soil also reduces. Also, as the relative stiffness of the pipe decreases, and due to the soil arching effects, the pipe attracts less load and the soil attracts more load. This phenomenon can be visualized by plotting the contact pressure for the 34” diameter pipe section as shown in Fig. II-14. As can be seen, the normal contact pressure acting on the pipe section reduces as the wall thickness is reduced, indicating that the pipe is attracting less load. As the soil attracts greater portion of the load, the amount of soil slippage increases, reducing the resulting hoop stresses.
Fig. II-13. Effect of wall thickness on longitudinal stresses in a 34” diameter pipe.

(a) Wall thickness of 5/8”  (b) Wall thickness of 1/16”

Fig. II-14. Effect of wall thickness on the contact pressure acting on a 34” diameter pipe.

Fig. II-15 shows the variation of the resulting stresses with the diameter-to-thickness ratio, $D/t$. As can be seen, the resulting stresses begin to increase rapidly as the $D/t$ ratio decreases below 100. The $D/t$ ratio is approximately 90 for the 34” pipe section and 53 for the 20” pipe section. As a result, the initial conclusions of increased local stiffness offsetting the increase in stresses of smaller pipe diameters during the burial depth analysis (Subsection 3.4.1) are confirmed.
Fig. II-15. Effect of diameter-to-thickness ratio, D/t, on the stresses at the crown of pipes of different diameters.

Lastly, examining the effects of wall thickness on pipe ovalization, Fig. II-16 shows that the degree of pipe ovalization increases as the wall thickness decreases. The increase is nearly linear, due to the reduction in pipe section stiffness. The increase begins to plateau at very low diameter-to-thickness ratio due to increased soil slippage and reduction in the load attracted by the pipe section.

Fig. II-16. Vertical ovalization of pipes of different diameters and wall thicknesses.
3.4.3. Effects of soil stiffness

Similar to studying the effects of wall thickness, analysis of the effect of soil stiffness was conducted using a 1.0 m depth of soil cover. The soil stiffness was varied between 5 MPa (very soft clay) and 250 MPa (dense sand/gravel). This covers a wide spectrum of possible backfill soil materials.

Fig. II-17 shows that as the soil stiffness is reduced, both the longitudinal and hoop stresses at the pipe crown increase exponentially. This increase in the stresses occurs as expected and is directly related to the soil arching effects. As the soil-to-pipe stiffness decreases, the pipe section attracts a larger portion of the load, increasing the resulting stresses. Again, smaller pipe diameters typically experience larger stresses. However, as the soil stiffness reduces to very small values, local bending effects begin to govern, and larger pipe diameters experience larger stresses.

![Graphs showing the effect of soil stiffness on stresses at the crown of pipes of different diameters.](image)

**Fig. II-17. Effect of soil stiffness on the stresses at the crown of pipes of different diameters.**

The degree of pipe ovalization also increases exponentially with the decrease in soil stiffness (Fig. II-18). As the soil stiffness is reduced, not only does the pipe attract a greater portion of the load, but also the passive confining resistance of the surrounding soil reduces. With a reduced confining pressure, the pipe is able to mobilize the surrounding soil a greater amount, and the overall ovalization of the section increases. As a result, there is a very rapid increase in pipe ovalization with very low soil stiffness. In soils with high stiffness, the soil itself resists a large portion of the load, the confining pressure is maximum, and the resulting stresses and pipe ovalization becomes negligible.
In each of the parametric studies discussed above, it was assumed that the Alberta CL-800 truck was travelling perpendicular to the pipe section. It was determined that since the pipe is continuously supported by the surrounding soil, the critical stresses would occur when the maximum axle load is placed directly above the pipe section. With the truck travelling perpendicular to the pipe axis, both wheel loads of the heaviest axle have the shortest load path to the pipe section, hence increasing the bending stresses.

However, it is also practical to assume that the truck load is travelling parallel to the pipe axis. In this situation, the pipe will be subjected to multiple concentrated loads along the length of the section (Fig. II-19). Due to the soil arching effects, it is expected that the resulting stresses are lower than the previous cases. Within this scenario, there exists two loading cases: one with the truck longitudinal axis centered with the pipe section (i.e. one-wheel line on either side of the pipe; Fig. II-20a) and one with a single wheel line centered over the pipe section (i.e. the truck longitudinal axis offset from the pipe section; Fig. II-20b).

The resulting normal stresses at the pipe crown as well as the vertical ovalization were obtained at the location of each axle for the 34” pipe section under each of the truck positions (Fig. II-20) and listed in Table II-2. It can be seen from the table that the critical case is with one wheel line located
directly over the pipe section (Fig. II-20b), with stresses nearly doubling in all cases, and ovalization increase by approximately 30%.

Fig. II-19. Alberta CL-800 truck travelling parallel to the pipe axis.

(a) Concentric truck               (b) Offset truck

Fig. II-20. Possible positions of truck travelling parallel to the pipe axis.
Table II-2. Stresses and pipe ovalization resulting from CL-800 truck travelling parallel to the pipe axis.

<table>
<thead>
<tr>
<th>Pipe Dia. (in.)</th>
<th>Truck Position</th>
<th>Axle Number</th>
<th>Maximum Stress (MPa) - directly below wheel</th>
<th>Vertical Ovalization (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pipe Crown</td>
<td>Springline</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Longitudinal</td>
<td>Hoop</td>
</tr>
<tr>
<td>34</td>
<td>Concentric</td>
<td>5</td>
<td>-13.12</td>
<td>-10.94</td>
</tr>
<tr>
<td></td>
<td>Offset</td>
<td></td>
<td>-23.00</td>
<td>-26.58</td>
</tr>
<tr>
<td>34</td>
<td>Concentric</td>
<td>4</td>
<td>-11.87</td>
<td>-11.53</td>
</tr>
<tr>
<td></td>
<td>Offset</td>
<td></td>
<td>-23.14</td>
<td>-29.28</td>
</tr>
<tr>
<td>30</td>
<td>Concentric</td>
<td>3</td>
<td>-14.12</td>
<td>-12.62</td>
</tr>
<tr>
<td></td>
<td>Offset</td>
<td></td>
<td>-22.54</td>
<td>-28.97</td>
</tr>
</tbody>
</table>

Notes: 1) Pipe wall thickness = 3/8”; 2) Burial depth = 1.0 m; 3) Soil stiffness = 25 MPa

The increase in stresses and ovalization is due to both the soil arching effects and the critical burial depth used. This study clearly showcases the importance of the proximity of the load to the pipe axis. The burial depth was taken as 1.0 m and the wheel spacing is 1.8 m for the Alberta CL-800 truck (Fig. II-6). Therefore, with the truck centered over the pipe axis, the direct load path is at nearly a 45° angle. It is common industry practice to assume a soil load transfer slope of 1H:2V, or 35°. It is clear in this case that much less load will be attracted to the pipe section and a major portion of the load will be resisted by the soil only.

With the truck offset from the pipe section (i.e. wheel line centered over the pipe axis), the vertical ovalization only increase by 30% rather than doubling similar to the stresses due to the confining pressure from the additional offset load. Although the second wheel line is 1.8 m away from the pipe section and has little contribution to the overall stresses, it does displace the soil towards to the pipe, providing an additional active confining pressure, limiting the overall vertical ovalization.

3.4.5. Effects of multiple trucks travelling perpendicular to the pipe axis

The effects of multiple truck loads travelling perpendicular to the pipe axis (Fig. II-21) were also analyzed. In this case, wheel loads of three parallel trucks were applied on the soil block assuming typical lane width of 3.75 m. The resulting stresses and pipe ovalization under the central truck...
were obtained and compared to the baseline case of a single truck discussed in Subsections 3.4.1 to 3.4.3. The comparison is presented in Table II-3.

![Image](image_url)

**Fig. II-21. Multiple trucks travelling perpendicular to the pipe axis.**

As can be seen in Table II-3, both the resulting stresses and ovalization increase by approximately 20 to 30%. As expected, the presence of the additional loading increases the stresses and ovalization at the center of the pipe section and also introduces additional stress concentrations along the length of the pipe, including negative bending moments as a result of the pipe being continuously supported (Fig. II-22). The resulting negative and positive bending moments indicates that under multiple truck loading, areas of the pipe section can be subjected to stress and strain reversals, which has the potential to result in fatigue failure should the stresses be large.

**Table II-3. Stresses and pipe ovalization resulting from multiple CL-800 trucks travelling perpendicular to the pipe axis.**

<table>
<thead>
<tr>
<th>Number of Trucks</th>
<th>Maximum Stress (MPa) - directly below wheel</th>
<th>Vertical Ovalization (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pipe Crown</td>
<td>Springline</td>
</tr>
<tr>
<td></td>
<td>Longitudinal</td>
<td>Hoop</td>
</tr>
<tr>
<td>Single truck</td>
<td>-31.51</td>
<td>-36.61</td>
</tr>
<tr>
<td>Three trucks</td>
<td>-24.15</td>
<td>-41.96</td>
</tr>
<tr>
<td>Difference</td>
<td>7.36</td>
<td>-5.35</td>
</tr>
</tbody>
</table>

Notes: 1) Pipe diameter = 34"; 2) Pipe wall thickness = 3/8"; 3) Burial depth = 1.0 m; 4) Soil stiffness = 25 MPa
Fig. II-22. Zones of negative bending moments and local stress concentration under the effects of multiple Alberta CL-800 trucks travelling perpendicular to the pipe axis.

3.4.6. Effects of heavy farming or construction equipment

Pipelines are typically located in rural areas and within farmyards. Therefore, it is practical to assume that the pipeline would be subjected to surface loading differing from the standard Canadian CL-W truck. The CL-W truck is the governing maximum load for designing most highway and bridge systems, however, rural construction equipment can have much higher axle loads. For the purposes of this analysis, a large earth moving truck (Fig. II-7) was used as the surface live loading. This truck has an axle load of 567 kN, which is more than 2.5 times the maximum axle load of the Alberta CL-800 truck. Results of the analysis under the effects of this earth moving truck are listed in Table II-4.

Table II-4. Stresses and pipe ovalization resulting from a heavy construction truck travelling perpendicular to the pipe axis.

<table>
<thead>
<tr>
<th>Type of Truck</th>
<th>Maximum Stress (MPa) - directly below wheel</th>
<th>Vertical Ovalization (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pipe Crown</td>
<td>Springline</td>
</tr>
<tr>
<td>CL-800</td>
<td>-31.51</td>
<td>-36.61</td>
</tr>
<tr>
<td>Construction truck</td>
<td>-61.11</td>
<td>-67.59</td>
</tr>
<tr>
<td>Difference</td>
<td>-29.60</td>
<td>-30.98</td>
</tr>
<tr>
<td></td>
<td>93.9 %</td>
<td>84.6 %</td>
</tr>
</tbody>
</table>

Notes: 1) Pipe diameter = 34"; 2) Pipe wall thickness = 3/8"; 3) Burial depth = 1.0 m; 4) Soil stiffness = 25 MPa
As expected, the resulting stresses and pipe ovalization under the effects of the earth moving truck increased significantly in comparison to those produced by the CL-800 truck. The increase in longitudinal stresses was in the range of 84-109 % and in the hoop stresses in the range of 48-85 %, whereas the pipe ovalization increased by more than 260 %. The stress increase is not proportional to the 250 % increase in the maximum axle load. This is attributed to the simultaneous 57 % increase in wheel line spacing and the 400 % increase in the wheel contact area. As discussed previously, the location of the applied load is critical to the resulting stresses. Since the loads are placed further from the midpoint of the pipe section, classical statics indicates that the bending moment will be reduced. Further, by having a larger contact area, the load becomes more distributed, and the soil attracts a larger portion of the load due to the soil arching effects.

3.4.7. Effects of ultimate limit state load factors and load combinations

For each of the parameters investigated thus far, service level loads were used in order to obtain the corresponding stresses and deformations to be expected. However, all structural Canadian Design Codes state that ultimate limit states (ULS) analysis be used for determining structural failure. Therefore, a ULS combination of dead and truck loads as specified by the Canadian Bridge Code CSA-S6-14, was applied:

\[
ULS: 1.25DL + 1.7LL(1 + DLA) \quad (II-4)
\]

The analysis was performed for the effects of a single CL-800 truck travelling perpendicular to the pipe axis (Fig. II-8). The resulting stresses and deformations are compared to those produced by the serviceability limit state (SLS) loads in Table II-5.

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Maximum Stress (MPa) - directly below wheel</th>
<th>Vertical Ovalization (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pipe Crown</td>
<td>Springline</td>
</tr>
<tr>
<td>Longitudinal</td>
<td>Hoop</td>
<td>Longitudinal</td>
</tr>
<tr>
<td>SLS</td>
<td>-31.51</td>
<td>-36.61</td>
</tr>
<tr>
<td>ULS</td>
<td>-49.08</td>
<td>-54.86</td>
</tr>
<tr>
<td>Difference</td>
<td>-17.57</td>
<td>-18.25</td>
</tr>
<tr>
<td></td>
<td>55.8 %</td>
<td>49.9 %</td>
</tr>
</tbody>
</table>

Notes: 1) Pipe diameter = 34"; 2) Pipe wall thickness = 3/8"; 3) Burial depth = 1.0 m; 4) Soil stiffness = 25 MPa
As indicated in the table, the increase in longitudinal stresses was in the range of 54-73\% and in the hoop stresses in the range of 41-50\%, whereas the pipe ovalization increased by almost 50\%. Since the pipe stresses remain within the elastic range, increasing the load by a factor results in a near linear increase in the stresses. These ULS stresses should then be compared to the yield stress, reduced by an applicable material resistance factor. Assuming a material resistance factor of 0.85 for ductile steel, the stress still remains well below the reduced yield of 306 MPa. It should be noted, however, that the analysis was performed on an intact non-corroded pipe of 3/8” standard wall thickness. Corrosion, however, could lead to significant reduction in the pipe wall thickness, and hence critical increase in the stresses that may lead to failure, particularly under repeated traffic loading.

### 3.5. Summary and Conclusions

Finite element analysis of in-situ decommissioned pipelines subjected to surface live load was investigated. The effects of burial depth, pipe diameter, wall thickness, soil stiffness, and load conditions on the resulting stress state were obtained and discussed. The following primary conclusions were drawn:

1. As the burial depth decreases, both the resulting longitudinal and hoop stresses increase. The increase is exponential indicating burial depth is a significant factor influencing the load path and the resulting stress state.

2. The stresses become increasingly significant at depths of soil cover less than 1.0 m. At burial depths greater than 1.0 m, the dynamic load allowance factor reduces and remains constant at 0.1. Therefore, the stresses converge at a much lower stress value.

3. As the wall thickness is reduced, the resulting longitudinal and hoop stresses begin to increase. The longitudinal stresses increase due to a reduced moment of inertia and increased bending stresses. The hoop stresses initially increase and then begin to decrease as the load becomes more localized, the pipe attracts less load, and the degree of soil slippage increases.

4. As the soil stiffness decreases, the resulting longitudinal and hoop stresses both increase due to the soil arching effects. As the stiffness of the soil decreases relative to the pipe
section, the soil is further mobilized, and the pipe section attracts more load, increasing the stresses.

5. As the pipe diameter decreases, the resulting stresses typically increase. However, this increase is offset by a simultaneous reduction in the \( D/t \) ratio if the wall thickness is kept constant. There exists a transition depth of 0.5 m after which large pipe diameters experience larger stresses due to local bending of the pipe wall.

6. Pipe ovalization increases with the decrease in the burial depth, wall thickness, and soil stiffness for all pipe diameters.

7. The direction and location of the traffic load relative to the pipeline axis has an effect on the pipe critical stress level. Traffic in a direction perpendicular to the pipe axis produces more critical stresses than traffic in a direction parallel to the pipeline axis.

8. Multiple trucks can increase the stress level to 30% higher than that produced by a single truck. Stress reversal can take place in the parts of the pipeline that are near but not directly affected by the traffic loading.

9. Heavy farming and construction equipment can produce stresses up to 110% higher than the stresses produced by the standard highway truck loading.

Despite the increase in stresses with reduction of the burial depth, pipe diameter, and wall thickness, the resulting stresses remain well below the pipe yield stress. It is observed that at practically deep soil cover and standard wall thicknesses, decommissioned pipelines pose little risk of collapse due to typical surface live load applied immediately after decommissioning. However, ultimate limit state (ULS) factored load combinations should be used for analysis of decommissioned buried pipelines against failure.

Location and direction of maximum stresses change as the wall thickness of the pipe section changes. This is of great importance when examining the effects of local/pitting corrosion. The criticality of local/pitting corrosion geometry and location is highly dependent on the location of the maximum stresses. It is necessary for the above investigations to be extended to include the effects of local/pitting corrosion in determining the structural response and capacity of buried decommissioned pipelines subjected to surface live loads.
3.6. Work in Progress

The effects of local corrosion have yet to be fully investigated to explore the effects of both general pitting corrosion and isolated extensive wall thinning at the location of critical stresses. A feasibility analysis has been conducted to determine the practicality of using ABAQUS to assess local corrosion. Two methods of reducing the wall thickness are employed: nodal thickness mapping and shell-to-solid coupling (Fig. II-23). Nodal thickness mapping (Fig. II-23a) involves defining the thickness of the shell elements at each node location. The nodes within an isolated region can be equally reduced, representing extensive local corrosion, or the node thickness can follow a random distribution similar to pitting corrosion. In either case, ABAQUS assumes a linear slope between node thicknesses, eliminating any abrupt changes in geometry which may result in unintended stress concentrations.

![Fig. II-23. (a) Shell nodal thickness; (b) Sell-to-solid coupling.](image)

A second method of modelling extensive local corrosion is through shell-to-solid coupling (Fig. II-23b), where the region of interest is modelled with solid elements, while the rest of the pipe is modelled using standard shell elements. This method allows for accurate modelling of the precise geometry of the corrosion, including width, length, orientation, and corner grooves/fillets, while reducing computational demand.

In both cases, stress concentrations appear at the corroded areas. Fig. II-24 shows the results of a feasibility analysis of a locally corroded pipe subjected to a uniformly distributed load. The figure clearly shows the stress concentration in the locally corroded wall of the pipe. Both methods will be further verified and then incorporated into the model to investigate the effects of local corrosion and the associated geometric parameters of the corrosion patch.
Fig. II-24. Stress concentrations at reduced section due to uniformly distributed load (feasibility analysis only).
4. Soil Subsidence

4.1. Introduction

Durability relates to the ability of a pipe to withstand, to a satisfactory degree, long-term wear and deterioration during its service life. Prolonged corrosion of pipe could result in cracks, fractures, and openings that allow surrounding soils to migrate into the pipe, a process which can be significantly accelerated by ground water infiltration. This accompanying erosion of the soil forms cavities around the cracks and openings, which could propagate towards the ground surface, culminating in sinkholes [Sato and Kuwano, 2015; Indiketiya et al., 2017]. Sinkholes pose a great threat on surface facilities [Guerrero, et al., 2008]. Fig. III-1 schematically illustrates the evolutionary development of a sinkhole due to internal soil erosion triggered by pipe deterioration. Among the potential consequences associated with sinkholes, ground subsidence is the most notorious and hazardous (Fig. III-2). Before becoming sufficiently large to produce sinkholes visible at the ground surface, erosion-induced void also threatens the stability and integrity of the pipelines due to loss of soil support and constraint around the pipe sections [Moore, 2008] (Fig. III-3).

![Fig. III-1. A schematic illustration of formation of sinkhole caused by soil erosion around pipe defects [Sato and Kuwano, 2015].](image-url)
Fig. III-2. Surface sinkhole and deteriorated steel pipes in Japan [Renuka, 2012].

Fig. III-3. Erosion voids in the vicinity of joints of sewer pipes [Balkaya et al., 2012].

To explore the problems associated with soil erosion due to defects in corroded pipes, one need to characterize behavior of eroded soil, soil erosion and migration mechanisms.
4.2. Characterization of Eroded Soil

4.2.1. Bingham model and the yield stress

Slurries of clayey soils usually display complex strain-rate- and time-dependent non-Newtonian flow behavior [Jeong et al., 2010]. Rheology is a field of study focusing on the characteristics and deformation of flow-able materials. One of the most popular rheological model characterizing clays’ visco-plastic flows is the Bingham model, which is defined by Eq. (III-1) [Barnes, 1999].

\[
\tau = \tau_y + \eta \dot{\gamma}
\]

(III-1)

where \( \tau \) is the shear stress, \( \tau_y \) is the yield stress, \( \eta \) is the viscosity, and \( \dot{\gamma} \) is the shear rate.

According the Bingham model, the yield stress is an important rheological parameter influencing the mobility of slurry flows. It refers to the shear stress causing the material to flow like a viscous fluid. Determination of the yield stress makes it possible to accurately predict flow characteristics of clay slurries. Therefore, it is necessitous to develop a viable yet simple, economical method to determine the yield stress in the laboratory.

4.2.2. Slump tests

Slump test originates as a testing method for determining the consistency (work-ability) of concrete. Too large the ‘slump height’ would cause the concrete to be unduly runny; too small the ‘slump height’ would result in unfavorably stiff concrete. Slump test has also been adopted into minerals industry for testing the consistency of a variety of inelastic fluids and mineral tailing suspensions [Gawu and Fourie, 2004].

The first analytical model relating the slump test to concrete’s yield stress was presented by Murata [Murata, 1984]. In geotechnical context, it was proposed that slump test can be used as a convenient method (rather than using the vane technique) to determine the yield stress of Bingham-
like geo-materials such as clays, debris, and sludges. A static model was developed to explain the deformation mechanism of geo-materials during a slump test using a truncated cone.

Slump tests can be performed under two geometry conditions: the conical test and the cylindrical test. By verifying against the results of yield stress obtained using the vane technique, Clayto et al. [Clayto et al., 2003] drew the conclusion that the cylindrical test is mathematically and experimentally superior to conical test.

4.2.3. Theoretical background

The analytical model for cylindrical slump test was initially developed by Pashias et al. [Pashias et al., 1996] and re-visited by Clayto et al. [Clayto et al., 2003] (Fig. III-4). For cylindrical slump tests, the relationship between the slump height and the material’s yield stress can be expressed by Eq. (III-2). It should be noted that this analytical model was derived with variables expressed in dimensionless form, therefore, this model is not empirical and it provides a material-independent relationship between yield stress and slump height.

$$s' = 1 - 2\tau_y' \left[ 1 - \ln(2\tau_y') \right]$$

(III-2)

where $s' = s/H$, dimensionless slump height, $s$ is actual slump height measured in the test, $\tau_y' = \tau_y/\rho g H$, dimensionless yield stress, and $\rho$ is density of the material tested.

Fig. III-4. Schematic diagram of the cylindrical slump test showing the initial and final stress distributions [Clayto et al., 2003].

In this study, cylindrical slump tests have been conducted on clayey soil slurries including sand, Regina Clay, Calgary till, and kaolinite. The aforementioned analytical model was adopted to investigate the soils’ yield stresses under varying water content conditions. Additionally, soils’
flow-abilities have been evaluated by examining their *run-out distances* during the tests for a more comprehensive understanding of their rheological behavior.

### 4.2.4. Materials

Four (4) types of soil were tested in this study: Ottawa sand, Regina clay, Calgary till and kaolinite. Soil properties were determined by laboratory tests, which are presented in Table III-1. The specific gravity was measured by following the ASTM standard D 854-14 [ASTM, 2014]. The angle of repose was determined by a hopper flow measurement of the oven-dried sand. The minimum porosity was tested according to the ASTM standard D 4254–16 [ASTM, 2016]. Liquid and plastic limits were determined according to ASTM standard D4318-17 [ASTM, 2017]. The particle size distribution of the sand is plotted in Fig. III-5. The measured repose angle and minimum porosity value are 34° and 37 %, respectively.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Specific Gravity</th>
<th>Liquid limit</th>
<th>Plastic limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>2.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Regina clay</td>
<td>-</td>
<td>71.3%</td>
<td>30.1%</td>
</tr>
<tr>
<td>Calgary till</td>
<td>-</td>
<td>32.8%</td>
<td>22.9%</td>
</tr>
<tr>
<td>Kaolinite</td>
<td>2.65</td>
<td>63.8%</td>
<td>41.2%</td>
</tr>
</tbody>
</table>

Table III-1. Soil properties of Regina clay, Calgary till and kaolinite.
Fig. III-5. Particle size distribution of the sand.

4.2.5. Testing procedure

The procedures of the tests are schematically illustrated in Fig. III-6. A plastic cylinder with an aspect ratio of 1.0 (84 mm in diameter and 84 mm in height) was used. Pashias et al. [Pashias et al., 1996] suggested that the aspect ratio should be around 1.0. The material slumps under its own weight during a test. Too large an aspect ratio would result in the collapse of the cylinder rather than the material flowing; too small would result in excessively small degree of slump.

In each test, 400 ± 20g dry soil was mixed with corresponding amount of water to form slurries. The slurry was then poured slowly into the cylinder until it was filled with slurry. Care was taken to prevent the formation of air bubbles. Next, the cylinder was slowly lifted up by hand and the slurry was allowed to slump. It was experimentally demonstrated that the rate of lifting-up has little influence on the slump height. Finally, the change in height between the cylinder and the deformed slurry (i.e. the slump height) was measured. Due to the fact that the top surface of the deformed slurry is concave in shape, a middle point of the concave surface was selected as the height. All the slump tests were conducted on a flat table. Pashias et al. (1996) showed that the basal friction did not affect the resulting slump height. In their tests, no measurable difference of slump height was found by testing smooth wood, rough wood, and stainless steel as the surface on which the material slumps. In addition, scales were drawn on the flat table, which allowed direct measurements of the run-out distance of the deformed soil slurries. The run-out distances were used to evaluate the soil mobility.
Details of slump tests are summarized in Table III-2. Densities of soil slurries were determined using a graduated cylinder and digital scale. A total of 15 tests have been performed using Regina Clay, Calgary Till, and kaolinite with varying water contents. Apart from the aforementioned tests, a series of slump tests on dyed soil samples were performed (Table III-3). At the end of each slump test, the slumped sample was allowed to freeze at a temperature of -5 °C in a cold room. Dyed soil slurries were used to allow visualization of soil movement. The objective was to investigate the variation of deformation characteristics of soil slurries with varying water contents. An individual test using dry sand was conducted under room temperature as a control sample to contrast with the results of clayey slurries.

Fig. III-6. Schematic illustration of cylindrical slump test.
Table III-2. Details of cylindrical slump tests.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Soil type</th>
<th>Water content (%)</th>
<th>Slurry density (g/cm³)</th>
<th>Liquidity index</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC_1</td>
<td>Regina clay</td>
<td>70</td>
<td>1325</td>
<td>0.97</td>
</tr>
<tr>
<td>RC_2</td>
<td></td>
<td>75</td>
<td>1309</td>
<td>1.09</td>
</tr>
<tr>
<td>RC_3</td>
<td></td>
<td>80</td>
<td>1295</td>
<td>1.21</td>
</tr>
<tr>
<td>RC_4</td>
<td></td>
<td>85</td>
<td>1255</td>
<td>1.33</td>
</tr>
<tr>
<td>RC_5</td>
<td></td>
<td>90</td>
<td>1224</td>
<td>1.45</td>
</tr>
<tr>
<td>CT_1</td>
<td>Calgary till</td>
<td>40</td>
<td>1535</td>
<td>1.73</td>
</tr>
<tr>
<td>CT_2</td>
<td></td>
<td>45</td>
<td>1499</td>
<td>2.23</td>
</tr>
<tr>
<td>CT_3</td>
<td></td>
<td>50</td>
<td>1456</td>
<td>2.74</td>
</tr>
<tr>
<td>CT_4</td>
<td></td>
<td>55</td>
<td>1397</td>
<td>3.24</td>
</tr>
<tr>
<td>CT_5</td>
<td></td>
<td>60</td>
<td>1351</td>
<td>3.75</td>
</tr>
<tr>
<td>K_1</td>
<td>Kaolinite</td>
<td>85</td>
<td>1454</td>
<td>1.94</td>
</tr>
<tr>
<td>K_2</td>
<td></td>
<td>90</td>
<td>1359</td>
<td>2.16</td>
</tr>
<tr>
<td>K_3</td>
<td></td>
<td>95</td>
<td>1274</td>
<td>2.38</td>
</tr>
<tr>
<td>K_4</td>
<td></td>
<td>100</td>
<td>1250</td>
<td>2.60</td>
</tr>
<tr>
<td>K_5</td>
<td></td>
<td>105</td>
<td>1205</td>
<td>2.82</td>
</tr>
</tbody>
</table>
Table III-3: Details of slump tests with dyed soil samples.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Soil type</th>
<th>Water content (%)</th>
<th>Test condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>DS_C</td>
<td>dry, dyed sand</td>
<td>-</td>
<td>20 °C, control sample</td>
</tr>
<tr>
<td>K_F_1</td>
<td>dyed kaolinite</td>
<td>80</td>
<td>-5 °C</td>
</tr>
<tr>
<td>K_F_2</td>
<td></td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>K_F_3</td>
<td></td>
<td>110</td>
<td></td>
</tr>
<tr>
<td>SK_F_1</td>
<td>15% sand mixed with 85% kaolinite (% by weight)</td>
<td>24</td>
<td>-5 °C</td>
</tr>
<tr>
<td>SK_F_2</td>
<td></td>
<td>26</td>
<td></td>
</tr>
</tbody>
</table>

4.2.6. Slump test results and discussion

**Slump height.** Fig. III-7 illustrates typical behavioral results of slump tests on Regina clay samples with varying water contents. Fig. III-8 shows the variation of slump heights with water content during the tests. It can be seen that there was a significant transition of slump height for Regina Clay (blue dots) when the water content was between 80 %-85 %. By comparison, the transitional water content for Calgary Till (red dots) lied between 45 %-50 %. However, kaolinite did not show a ‘watershed’ water content: the slump height showed no sign of cessation as the water content increased (green dots).
(a) Regina Clay with 70% water content.

(b) Regina Clay with 75% water content.
(c) Regina Clay with 85% water content

Fig. III-7. Results of slump tests on Regina clay sample with varying water content.

Fig. III-8. Slump height measured during the tests.
**Yield stress.** Utilizing the slump heights obtained in the tests, yield stresses of slurries were able to be calculated by Eq. (III-2). Fig. III-9 shows the relationship between the yield stress and water content. Empirical relationships were obtained based on the observational results. For Regina Clay, the yield stress did not show significant further decrease after the water content reached 85 %. The threshold water contents for Calgary Till and kaolinite were 50 % and 100 %, respectively.

![Graph showing the relationship between yield stress and water content.](image)

**Fig. III-9.** Yield stress of soil slurries.

![Graph showing the relationship between liquidity index and yield stress.](image)

**Fig. III-10.** Relationship between liquidity index and yield stress.
Fig. III-10 shows correlation between liquidity index and yield stress. Compared with Regina clay and kaolinite, yield stress of Calgary till (red triangles) exhibited higher sensitivity to the change of liquidity index. This is particularly true for a liquidity index greater than 2.5.

**Soil mobility.** Fig. III-11 shows the run-out distance in each test. In general, run-out distances increased with increasing water content for all soil slurries. The average radial run-out distance in each test was also plotted against the corresponding water content, as shown in Fig. III-12. Empirical relationships were obtained based on the observational results during the tests. It is interesting to observe that all types of soil display approximately linear relationship between the average run-out distance and water content.
Fig. III-11. Soil mobility observed in the test.

Fig. III-12. Variation of average radial run-out distance with varying water contents.
Characteristics of deformation. From Fig. III-13, it was observed that the sand deformed in a layer-by-layer fashion. Soil interferences between different layers were not significant; each layer of sand formed a sand dune. The resultant angle of response was close to 33°.

Fig. III-13: Internal deformation characteristics of dry sand.

In terms of the internal deformation characteristics of soil slurries, at a relatively low water content (80%) the kaolinite slurry also deformed in a layer-by-layer fashion (Fig. III-14). Each soil layer stacked on top of each other, showing non-yielding behavior. However, as the water content increased, the deformation behavior transitioned into an intrusive manner. As shown in Fig. III-
15, at a water content of 100%, the bottom yellow layer yielded and it was squeezed toward the periphery of the deformed slurry. The two layers in the middle (green and red) also behaved in the same way. The top blue layer did not yield due to its own weight. With further increase of water content (Fig. III-16), the top blue layer yielded and the whole sample deformed as a viscous fluid.

Fig. III-15. Internal deformation characteristics of kaolinite slurry (water content = 100%).

As shown in Figs. III-17 and III-18, two test cases were performed using sand-kaolinite mixture. At a water content of 24%, layer-by-layer deformation was observed. Only the bottom layer yielded. When the water content was increased to 26%, three lower soil layers all exhibited yielding behavior.

Fig. III-16. Internal deformation characteristics of kaolinite slurry (water content = 110%).
Fig. III-17. Internal deformation characteristics of sand kaolinite mixture (water content = 24 %).

Fig. III-18. Internal deformation characteristics of sand kaolinite mixture (water content = 26 %).

Using the empirical relationship proposed by Locat [Locat, 1997], the viscosity for soft clays in the range of liquidity indices 1–5 can be obtained by Eq. (III-3). The results are summarized in Table III-4 and Fig. III-19.
\[ \eta = 0.52(\tau_y)^{1.12} \]  

(III-4)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Soil type</th>
<th>Water content (%)</th>
<th>Liquidity index</th>
<th>Yield stress (Pa)</th>
<th>Viscosity (mPa·s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC_1</td>
<td>Regina clay</td>
<td>70</td>
<td>0.97</td>
<td>303.2</td>
<td>-</td>
</tr>
<tr>
<td>RC_2</td>
<td></td>
<td>75</td>
<td>1.09</td>
<td>134.0</td>
<td>125.4</td>
</tr>
<tr>
<td>RC_3</td>
<td></td>
<td>80</td>
<td>1.21</td>
<td>111.9</td>
<td>102.5</td>
</tr>
<tr>
<td>RC_4</td>
<td></td>
<td>85</td>
<td>1.33</td>
<td>23.6</td>
<td>17.9</td>
</tr>
<tr>
<td>RC_5</td>
<td></td>
<td>90</td>
<td>1.45</td>
<td>19.7</td>
<td>14.7</td>
</tr>
<tr>
<td>CT_1</td>
<td>Calgary till</td>
<td>40</td>
<td>1.73</td>
<td>293.5</td>
<td>301.8</td>
</tr>
<tr>
<td>CT_2</td>
<td></td>
<td>45</td>
<td>2.23</td>
<td>137.3</td>
<td>128.9</td>
</tr>
<tr>
<td>CT_3</td>
<td></td>
<td>50</td>
<td>2.74</td>
<td>46.2</td>
<td>38.0</td>
</tr>
<tr>
<td>CT_4</td>
<td></td>
<td>55</td>
<td>3.24</td>
<td>36.9</td>
<td>29.6</td>
</tr>
<tr>
<td>CT_5</td>
<td></td>
<td>60</td>
<td>3.75</td>
<td>29.0</td>
<td>22.6</td>
</tr>
<tr>
<td>K_1</td>
<td>Kaolinite</td>
<td>85</td>
<td>1.94</td>
<td>228.8</td>
<td>228.3</td>
</tr>
<tr>
<td>K_2</td>
<td></td>
<td>90</td>
<td>2.16</td>
<td>181.2</td>
<td>175.8</td>
</tr>
<tr>
<td>K_3</td>
<td></td>
<td>95</td>
<td>2.38</td>
<td>158.6</td>
<td>151.5</td>
</tr>
<tr>
<td>K_4</td>
<td></td>
<td>100</td>
<td>2.60</td>
<td>61.7</td>
<td>52.6</td>
</tr>
<tr>
<td>K_5</td>
<td></td>
<td>105</td>
<td>2.82</td>
<td>43.5</td>
<td>35.6</td>
</tr>
</tbody>
</table>

Table III-4. Viscosity of soil slurries calculated based on Eq. (III-3).
In general, viscosity of Calgary till (red triangles) exhibited higher sensitivity to the change of liquidity index.

4.2.7. Numerical modelling of slump tests

Dry sand samples. Fig. III-20 shows the results of a slump test on dyed dry sand in sequential time. The commercial software pfc-3d was used to simulate the experimental results. The program is written based on discrete element method. 1,600 elemental balls (diameter = 2.13 - 2.84 mm) were used. The diameter of sand grain in the slump test is around 0.3 mm. If using that, the number of elemental balls used in the simulation is more than 20,000 elemental balls, which is extremely computational demanding. The physical and mechanical input parameters used are listed in Table III-5.

All the balls are generated in a way such that the porosity = 0.385 when lifting the cylindrical wall, which is comparable to that for sand used in the slump test. 12800 calculation steps were used in each case. Note that the friction coefficient of ball surface is not the repose angle.
Table III-5. Important parameters used in the PFC simulation.

<table>
<thead>
<tr>
<th>Elemental balls</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of balls</td>
<td>1600</td>
</tr>
<tr>
<td>Density (kg/m³)</td>
<td>2600</td>
</tr>
<tr>
<td>Diameter of balls (mm)</td>
<td>2.7 – 4.05</td>
</tr>
<tr>
<td>Normal stiffness (N/m)</td>
<td>1e3</td>
</tr>
<tr>
<td>Shear stiffness (N/m)</td>
<td>1e3</td>
</tr>
<tr>
<td>Friction coefficient of ball surface and wall</td>
<td>0.15</td>
</tr>
<tr>
<td>Normal stiffness (N/m)</td>
<td>1e4</td>
</tr>
<tr>
<td>Shear stiffness (N/m)</td>
<td>1e4</td>
</tr>
<tr>
<td>Friction coefficient between ball and wall</td>
<td>0.25</td>
</tr>
</tbody>
</table>

The PFC simulation results are presented in Fig. III-21 for different cases. They are comparable to those observed in tests. The response angle of the sand pile increases with increasing friction coefficient of ball surface.
Figure III-20. Pictures showing stages in a slump test on dyed dry sand.
Side view 3D view

**Case 1**: Friction coefficient of ball surface = 0.0

Side view 3D view

**Case 2**: Friction coefficient of ball surface = 0.15

Side view 3D view

**Case 3**: Friction coefficient of ball surface = 0.5

Fig. III-21. PFC simulation results of a slump test on dry sand with different ball surface friction coefficient.
Clay samples. Program flac was used in the simulation. The clay sample was assumed to behave linear isotropic viscoelastic (Table III-6). A mesh of a cylindrical sample was generated and allowed to reach an initial state of equilibrium (using elastic model). Then, the material was allowed to deform under its own weight. This process mimics the physical deformation during the slump test. To mimic the flow behavior, time-dependent deformation analysis was conducted, as shown in Fig. III-22.

Table III-6. Important parameters of viscoelastic model used in the FLAC simulation.

<table>
<thead>
<tr>
<th>Viscoelastic model</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk modulus (Pa)</td>
<td>6e4</td>
</tr>
<tr>
<td>Shear modulus (Pa)</td>
<td>6e3</td>
</tr>
<tr>
<td>Viscosity (Pa·s)</td>
<td>1e3</td>
</tr>
<tr>
<td>Density (kg/m³)</td>
<td>1500</td>
</tr>
<tr>
<td>Time-step in creep simulation (s⁻¹)</td>
<td>1e-3</td>
</tr>
</tbody>
</table>

Fig. III-22. FLAC simulation results of a slump test on clay sample.
4.2.8. Summary

Slump test is able to be utilized as a practical, consistent, and convenient method to determine the yield stress of clayey soil slurries whose flow behavior can be characterized by the Bingham model. This study elaborates the laboratory testing procedures of the determination of yield stresses for Regina Clay, Calgary Till, and kaolinite slurries with varying water contents. The characteristics of soil slurry deformation were also qualitatively discussed. The following conclusions can be drawn:

(1) It was found that there existed a threshold water content above which no further decrease in yield stress occurred, irrespective of soil type.

(2) Yield stress and viscosity of Calgary till showed comparatively high sensitivity to the change of liquidity index.

(3) Soil mobility was also evaluated base on the observational run-out distances during the slump tests. For all soil slurries, the average run-out distances were linearly related to their water contents.

4.2.9. Practical implications

This study is focused on the determination of yield stress for clayey soil slurries whose flow behavior can be characterized by Bingham rheological model. Under the field condition, rain infiltration and groundwater flow could trigger slurry flow through defects on the deteriorated pipes, causing ground loss and other related problems. Numerical models for characterizing slurry flow mechanism will require input of rheological parameters such as yield stress and viscosity [Jeong et al., 2010]. Results of this study provide a practical solution for obtaining yield stress of clayey soil slurries.

More broadly, debris flows are multi-phase, gravity-driven flows consisting of randomly dispersed interacting phases [Pudasaini, 2010]. Complete understanding of the rheological behavior of debris flow still remains extremely challenging. The results of this study can be applied to the liquid phase of debris flow.
4.3. Physical Modeling: Soil Mobility due to Pipeline Corrosion

4.3.1. Rationale and test setup

The objective of the physical model test is to investigate the soil run-out distance and ground subsidence due to pipeline corrosion. As shown in Fig. III-23, a plastic pipe was placed beneath a wood box. A hole with a diameter of 1.5 cm was drilled on the wall of the pipe, which was used to mimic the corrosion damage. The wood box was filled with sandy soil with a height of 30 cm. Therefore, the plastic pipe could mimic as a section of pipeline buried underground. Initially a stopper was placed to prevent the soil from flowing into the pipe before the test. The stopper was removed as soon as the test began; the soil was allowed to flow into the pipe and ground subsidence developed.

Fig. III-23. Test setup of physical model test.
4.3.2. Test results

The test results are shown in Fig. III-24 sequentially. At end of the test, the sinkhole reached a diameter of 50.5 cm.
Fig. III-24. Results of physical model test on sand showing soil subsidence and migration into and along pipe.

4.4. Prediction of soil subsidence and migration into corroded pipe

4.4.1. Criteria for soil erosion around the corroded hole in the pipe section

To produce or erode soil from the soil above the corroded hole (opening), the water seepage force must be sufficiently high to de-stabilize the soil structure. Wong et al. [Wong et al., 1994] derived analytical solutions for different stress states which are listed below.

For the soil element in in situ stress equilibrium state, the normalized pressure gradient induced by water seepage is given by:

\[ \frac{dp}{\gamma_w dr} = \frac{\sigma_t}{a} \]  \hspace{1cm} (III-4)

where \( dp \) = differential pressure, \( dr \) = differential distance, \( \gamma_w \) = unit weight of water, \( \sigma_t \) = tangential stress acting on the soil element (= \( K_o \gamma_s H \) where \( K_o \) = in situ stress coefficient, \( \gamma_s \) = unit weight of bulk soil; \( H \) = soil cover), and \( a \) = opening radius.
For the soil element subjected to yielding, the normalized pressure gradient induced by water seepage is given by:

\[
\frac{dp}{\gamma_w dr} = N c (\cot (45^\circ - \frac{\varphi}{2}))
\]  

where \( N \) = shape factor (being 2 for long slot case and 4 for circular hole), \( c \) = soil cohesion, and \( \varphi \) = soil friction angle.

4.4.2 Pressure gradient due to water seepage around the corroded hole in the pipe section

The pressure gradient in the soil around the hole due to the water head above the pipe was analyzed using program SEEP/W. The program yields total head potential, pressure head contours, and flow lines. The results were used to calculate the pressure gradient around the hole and determine if the gradient is sufficient height to trigger soil erosion around the hole. In the analyses, the hole size was varied to assess the critical situations.

4.4.3 Estimation of soil subsidence above the corroded hole and soil migration into and along the pipe section

Fig. III-25 shows the interaction between soil subsidence above the corroded hole and soil migration into and along the pipe section. The amount of soil subsidence or erosion depends on the repose angle (\( \theta \)) of compacted soil above the pipe, the water seepage de-stabilizing the slope, and the soil pipe-up at the hole blocking the soil erosion. The amount of soil migration into and along the pipe section depends on the repose angle (\( \alpha \)) and density of eroded soil or soil slurry, pipe diameter and alignment. Results of slump tests on different soil types and water contents were used to estimate their repose angles and bulk densities.

Based on the mass balance criterion, the mass of soil subsidence is equal to that of soil erosion. Thus, the volume of soil subsidence, \( V_{sub} \) is related to that of soil migration \( V_{mig} \) into and along the pipe in the following equation:

\[
V_{mig} = \frac{\gamma_s}{\gamma_{slurry}} V_{sub}
\]  

where \( \gamma_s \) = unit weight of compacted soil, and \( \gamma_{slurry} \) = unit weight of eroded soil.
Fig. III-25. Schematic showing the relationship in soil volumes caused by soil subsidence above the corroded hole and soil migration into the pipe.

4.4.4 Results

Fig. III-26 shows the simulation results of SEEPW. This case represents the water seepage through a soil mass of 1 m in thickness above a buried pipe. The corroded hole has an opening of 30 cm in diameter. The water level is maintained at the surface. There is a 1-m total head between the hole location and surface. Fig. III-26a shows the pressure distribution in the soil mass with intervals of 0.05 m, and the flow lines. Fig. III-26b plots the equipotential contours of 0.1-m head intervals with flow lines. The pressure gradient near the hole was calculated as 2.04 for this 2D case. For 3D case, the pressure gradient is assumed to be increased by a factor of $\pi/2$. 
Fig. III-26. SEEP/W simulation results of water seepage above an opening in a corroded pipe section (a) pressure head contours and flow lines (b) total head contours and flow lines. Soil cover = 1m, opening size = 30 cm, water level at ground surface.

The above analysis was repeated with hole sizes in a range of 0.5 – 60 cm. The results are plotted in Fig. III-27. The critical pressure gradients required for soil erosion in different soil types and conditions were calculated from Eqs. (III-4) and (III-5) and include in Fig. III-27 for assessment. For the case of soil arch, $K_o = 0.5$, $H = 1$ m. and $\gamma_s = 20$ kn/m$^3$. For clayey soil, $c = 45$ kPa and $\phi = 0^\circ$. For sandy soil, $c = 1$ kPa and $\phi = 35^\circ$. It can be seen from Fig. III-27 that sand would be eroded when the hole size is greater than 10 cm. The 1-m head is not sufficient to cause any erosion in compacted clay.

The volumes of soil subsidence and soil erosion and migration into the pipe were calculated based on the configurations shown in Fig. III-25 with varying repose angles, slope angles and pipe alignment angles. The $\gamma_{slurry} = 15$ kn/m$^3$ was used. The results are plotted in Fig. III-28. The figure reveals that the volume of soil subsidence is much larger than that of soil migration into the pipe. The soil subsidence would be limited by the amount of space available for soil accumulated in the pipe. The soil subsidence would stop when the hole was fully blocked by the soil pile inside the
pipe. For 10-cm hole, the amount of soil subsidence varies in a range of 1 to 10 m$^3$ depending on the pipe alignment.

![Diagram](image)

**Fig. III-27.** Comparison of normalized pressure gradient as function of corroded hole radius for water seepage in 1-m soil cover and different conditions (in situ soil arch, sandy and clayey soils).

![Diagram](image)

**Fig. III-28.** Volumes of soil subsidence and soil migration into pipe section as function of soil repose angle for different soil covers of 1 and 2 m and pipe horizontal alignments of 0 and 5 degrees.
5. Theme IV: Model for Risk Assessment

5.1. Research background

Pipelines have been existent for over 150 years to carry essential energy resources such as oil, natural gas, and water. Pipelines can be buried underground or placed above the ground depending on the geologic terrain it traverses. However, most buried pipelines are typically in service for 50 years. Thereafter, the line becomes decommissioned and is replaced with a new line. New lines are built either because an efficient alternate route and right-of-way is proposed or there were existing problems with the decommissioned line such as the line being subjected to geohazards such as landslides or ground subsidence.

Decommissioned lines can be dealt with in two distinct ways. The first method can be to excavate the soil along the existing right-of-way of the decommissioned line and extract the pipe out and backfill the excavated trench. However, with the considerable effort and time required, this is not the most economical option. Thus, the second method in dealing with decommissioned lines is to leave the existing line intact. While the second method is the most economical option, there are concerns raised by property owners whose land coincides with the decommissioned line right-of-way. The main concern centres on the collapse of the decommissioned line leading to possible unstable ground patterns such as sinkholes which may cause potential property damage and safety to people.

This part aims to address the concerns regarding possible failure of a decommissioned pipeline as a result of loss of structural integrity due to external corrosion and soil overburden. The most appropriate approach to this problem is using a probabilistic-based risk and reliability method. In order to conduct such stochastic modelling, one need to define the limit state functions for the specified objectives.

5.2. Limit State Functions

A decommissioned pipeline is subjected to external corrosion. The external corrosion rate and magnitude depends on many factors such as pipe coating, pipe type, soil type and soil cover, temperature, and soil physical and chemical properties. One of major concerns is the external corrosion causing material loss in the pipe wall and formation of holes through the pipe wall. Soil
erosion and migration into the pipe will induce soil subsidence. Another major concern is the structural integrity of the corroded pipe under imposed loads. The imposed loads include soil overburden and traffic load. In this theme, two possible limit state functions will be investigated: (i) external corrosion leading to the loss of pipe wall thickness (pitting corrosion), and (ii) flexure yielding of corroded pipe section under soil overburden.

5.2.1. **Limit state function for pitting corrosion**

The limit state function for loss of pipe wall thickness (pitting corrosion) is simply the difference between the initial pipe wall thickness, \( w_0 \), and the corroding wall thickness with respect to time, \( w(t) \):

\[
LSF: w_0 - w(t) \leq 0
\]  
(IV-1)

There are many mathematical expressions that can be used to express the corroding wall thickness with respect to time, \( w(t) \). For this study, a general simple power-law formulation based on pitting corrosion will be used [Velazquez et al., 2009]:

\[
w(t) = \kappa t^\nu
\]  
(IV-2)

where \( \kappa \) is an equivalent corrosion rate multiplier. When \( \nu = 1 \), \( w(t) \) becomes linear and thus, \( \kappa \) is the corrosion rate itself. Thus, the final expression for the limit state function for loss of pipe wall thickness (pitting corrosion) is:

\[
LSF: w_0 - \kappa t^\nu \leq 0
\]  
(IV-3)

5.2.2. **Limit state function for flexure yielding**

The limit state function for flexure yielding is expressed as function of the critical bending stress on the pipe, \( \sigma_b \), and the ultimate strength of the pipe steel, \( \sigma_u \):

\[
LSF: \sigma_u - \sigma_b \leq 0
\]  
(IV-4)

The ultimate strength of the pipe steel is well defined in published literature on pipelines and steel properties in general. The critical bending (tensile) stress due to the soil overburden on the pipe for external corrosion is located on the spring line (Fig. IV-1) as:
\[
\sigma_b = \frac{M_d \gamma}{l}
\]  \hspace{1cm} (IV-5a)

with:

\[
M_d = \alpha \gamma H D^2
\]  \hspace{1cm} (IV-5b)

where \(\alpha\) is a coefficient based on an equivalent beam with end support conditions, \(\gamma\) is the soil unit weight, \(H\) is the pipe burial depth, and \(D\) is the pipe diameter. The flexural capacity of the pipe with a uniform corroded section, \(M\), can be expressed in terms of \(\sigma_u\) as:

\[
M = \frac{\sigma_u l_u}{\bar{y}_u} = \frac{\sigma_u w^2}{6}
\]  \hspace{1cm} (IV-6)

where \(w\) is the remaining intact wall thickness of the pipe. However, with the pipe being subjected to pitting corrosion rather than uniform corrosion, the flexural capacity of the pipe, \(M\), can be multiplied by an adjustment factor accounting for pitting corrosion, \(\beta\). Fig. IV-1 illustrates the location of the critical bending stress on the pipe, and Fig. IV-2 illustrates the dimensions \(H\) and \(D\) of the buried pipe. Thus, the final expression for the limit state function for flexure yielding is:

\[
LSF: \beta M - M_d \leq 0
\]  \hspace{1cm} (IV-7)

---

Fig. IV-1. Illustration of the critical bending section location.
Fig. IV-2. Illustration of burial depth, H, and pipe diameter, D. The burial depth, H, is measured from the ground surface to the pipe spring line.

5.3. Probability Simulation Process

Determination of the expected life time of a decommissioned pipeline was completed by running a Monte-Carlo simulation of 100,000 events. The 100,000 events represent a simulation of every 100,000 pipeline segments. The failure probability, $P_f$, for a given time, $t$, is simply the number of failure events (i.e., segment failures) divided by the total number of events.

$$P_f = \frac{\text{Number of Failures}}{\text{Total Number of Events}} \quad (\text{IV-8})$$

The number of failure events is the number of events where the limit state function of interest reaches either zero or a negative value. The critical value for $P_f$ in structural reliability is $10^{-4}$ or one failure per 10,000 events.

5.3.1. Simulation parameters for pipe wall thickness loss (pitting corrosion)

The limit state function for pipe wall thickness loss (pitting corrosion) is given by Eq. (IV-3). A probabilistic distribution was assigned to each of the parameters in the limit state function for pitting corrosion. Table IV-1 summarizes the probability distribution type and respective distribution values chosen for each variable.
Table IV-1. Probability distributions for each parameter in the pitting corrosion limit state function.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Probability Distribution Type</th>
<th>Mean Value</th>
<th>Coefficient of Variation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$w_0$</td>
<td>Constant Value</td>
<td>12.7 mm</td>
<td>0</td>
</tr>
<tr>
<td>$\kappa$</td>
<td>Log-Normal</td>
<td>0.164 mm/yr$^\nu$</td>
<td>30</td>
</tr>
<tr>
<td>$\nu$</td>
<td>Uniform</td>
<td>0.7815</td>
<td>6.1</td>
</tr>
</tbody>
</table>

5.3.2. Simulation parameters for flexure yielding

Combining Eqs. (IV-2) and (IV-4) – (IV-6), the limit state function for flexure yielding becomes:

$$L SF: \sigma_u - \frac{C_o y H}{\beta} \left(\frac{D}{w_0 - \kappa t}\right)^2 \leq 0 \quad \text{(IV-9)}$$

where the corrosion rate follows the power law of Eq. (IV-3). Despite Eq. (IV-3) being based on pitting corrosion, $w(t)$ in Eq. (IV-9) is assumed as uniform corrosion since the effect of pitting corrosion is described using $\beta$ (i.e., if $\beta = 1$, Eq. (IV-9) represents uniform corrosion).

A probabilistic distribution was assigned to each of the parameters in the limit state function above. Table IV-2 summarizes the probability distribution type and respective distribution values chosen for each variable. The values of $\kappa$ and $\nu$ parameters are those suggested by Velazquez et al. [Velazquez et al., 2009].
Table IV-2. Probability distributions for each parameter in the flexural yielding limit state function.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Probability Distribution Type</th>
<th>Mean Value</th>
<th>Coefficient of Variation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \sigma_u )</td>
<td>Normal</td>
<td>360 - 760 MPa</td>
<td>3.3 - 6.9</td>
</tr>
<tr>
<td>( C_\alpha )</td>
<td>Log-normal</td>
<td>0.1878</td>
<td>20.3</td>
</tr>
<tr>
<td>( \gamma )</td>
<td>Constant value</td>
<td>19 kN/m(^3)</td>
<td>0</td>
</tr>
<tr>
<td>( H )</td>
<td>Constant value</td>
<td>1.381 m</td>
<td>0</td>
</tr>
<tr>
<td>( D )</td>
<td>Constant value</td>
<td>0.762 m</td>
<td>0</td>
</tr>
<tr>
<td>( \beta )</td>
<td>Constant Value</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>( w_0 )</td>
<td>Constant value</td>
<td>12.7 mm</td>
<td>0</td>
</tr>
<tr>
<td>(pitting corrosion)</td>
<td>Log-normal</td>
<td>0.164 mm/yr(^\nu)</td>
<td>30</td>
</tr>
<tr>
<td>( \nu ) (pitting corrosion)</td>
<td>Uniform</td>
<td>0.7815</td>
<td>6.1</td>
</tr>
<tr>
<td>(uniform corrosion)</td>
<td>Log-normal</td>
<td>0.2-160 (\mu)m/yr</td>
<td>30</td>
</tr>
</tbody>
</table>

Notes:

- The Coefficient of Variation is the ratio between the standard deviation and the mean value.
- Ranges of \( C_\alpha \) are given in literature [Einstein and Schwartz, 1979].
- The ultimate strength, \( \sigma_u \), is based on X52 pipe steel being used.
- \( H = 1 + D/2 \)
- Mean values and standard deviations of \( \kappa \) and \( \nu \) are estimated from the literature [Velazquez et al., 2009].
5.4. Results and discussion

5.4.1. Pitting corrosion

Based on the probability distributions listed in Table IV-1 and changing the time, \( t \), Fig. IV-3 illustrates a plot of the occurrence or “failure” probability with respect to time for the pitting corrosion limit state function based on the power law corrosion [Velazquez et al., 2009]. Fig. IV-3 can be replotted as an equivalent log-log plot as illustrated in Fig. IV-4. Using the reliability criterion, the critical probability, \( P_f \), of \( 10^{-4} \) is achieved at about 63 years for pitting corrosion.

![Graph showing the probability of failure vs. time for pitting corrosion.](image)

Fig. IV-3. Plot of probability with respect to time for the pitting corrosion limit state function based on the power law corrosion [Velazquez et al., 2009].
Fig. IV-4. Log-log plot of occurrence probability with respect to time for the pitting corrosion limit state function based on the power law corrosion [Velazquez et al., 2009].

5.4.2. Flexure yielding

The same exercise above can be repeated for the flexure yielding limit state function. Using the probability distributions listed in Table IV-2 and changing the time, $t$, Fig. IV-5 illustrates a plot of the occurrence probability with respect to time for the flexure yielding limit state function based on the power law corrosion [Velazquez et al., 2009] and $\sigma_u = 760$ MPa. Fig. IV-5 can be replotted as an equivalent log-log plot as illustrated in Fig. IV-6. Using the structural reliability criterion, the critical occurrence probability, $P_f$, of $10^{-4}$ is achieved at around 50 years for flexure yielding based on the power law corrosion [Velazquez et al., 2009].
Fig. IV-5. Plot of occurrence probability with respect to time for the flexural yielding limit state function based on power law corrosion [Velazquez et al., 2009] and $\sigma_u = 760$ MPa.

Fig. IV-6. Log-log plot of occurrence probability with respect to time for the flexural yielding limit state function based on the power law corrosion [Velazquez et al., 2009] and $\sigma_u = 760$ MPa.
A parametric study which examine the effect of certain parameters on the occurrence or “failure” probability versus time plot in the flexural yielding limit state functions of Eq. (IV-9).

**Power law exponent** \( \nu \). The exponent, \( \nu \), in the power law expression for the corroding wall thickness, \( \kappa t^\nu \), is found in both the flexure yielding limit state function. Fig. IV-7 illustrates the increase in the life expectancy of a decommissioned pipeline when the power law exponent, \( \nu \), drops from 1 to 0.7 with the flexural yielding limit state function for a constant \( \kappa \). The time it takes to reach a critical occurrence probability, \( P_f \), of \( 10^{-4} \), is 22 years, 50 years, and 90 years for \( \nu \) of 1, 0.8, and 0.7, respectively. Thus, having an accurate value for \( \nu \) is essential in having precise Monte-Carlo simulations in predicting the life expectancy of a decommissioned pipeline.

![Graph showing the increase in decommissioned pipe life expectancy with decrease in power law exponent. The corrosion rate multiplier, \( \kappa \), is kept at a mean value of 0.15 mm/yr\(^\nu\) with a coefficient of variation of 30% (log-normal distribution) for the plot. \( \sigma_u = 760 \) MPa.](image)

**Corrosion rate multiplier** \( \kappa \). The corrosion rate multiplier, \( \kappa \), in the power law expression for the corroding wall thickness, \( \kappa t^\nu \), is also found in both the flexural yielding limit state functions. For simplicity, a linear corroding wall thickness (i.e. \( \nu = 1 \)) was used to determine the influence of \( \kappa \) on...
the life expectancy of a decommissioned pipeline. Thus, $\kappa$ is the corrosion rate (mm/yr) itself. Fig. IV-8 illustrates the increase in the life expectancy of a decommissioned pipeline when the corrosion rate multiplier, $\kappa$, decreases from 1 mm/y down to 0.01 mm/y with the flexural yielding limit state function. Fig. IV-8 reaffirms this trend at a critical occurrence probability of $10^{-4}$. Once again, it is anticipated the trend in Fig. IV-8 will shift further to the right by repeating this exercise with the pitting corrosion limit state function. Thus, finding an appropriate value for $\kappa$ is also essential in having precise Monte-Carlo simulations. Table IV-3 lists representative values of corrosion rates for steel when subjected to atmospheric and soil conditions courtesy [American Galvanizers’ Association, 2018]. In fact, the American Galvanizers’ Association add that “With more than 200 different types of soil identified in North America, corrosion rate in soils is varied and hard to predict. Steel requires oxygen, moisture and the presence of dissolved salts to corrode. If any one of these is absent, the corrosion reaction will cease or proceed very slowly. Steel corrodes quickly in acidic environments and slowly or not at all as alkalinity is increased.”

![Graph](image)

**Fig. IV-8.** Flexure yielding - Increase in decommissioned pipe life expectancy with decrease in corrosion rate multiplier. The power law exponent, $\nu$, is kept at a constant value of 1 for the plot. $\sigma_u = 760$ MPa.
Table IV-1. Typical uniform corrosion rates for steel subjected to various environments [American Galvanizers' Association, 2018].

<table>
<thead>
<tr>
<th>Atmosphere (in air)</th>
<th>Corrosion Rate (μm/yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rural</td>
<td>4 - 60</td>
</tr>
<tr>
<td>Urban</td>
<td>30 - 70</td>
</tr>
<tr>
<td>Industrial</td>
<td>40 - 160</td>
</tr>
<tr>
<td>Marine</td>
<td>60 - 170</td>
</tr>
</tbody>
</table>

In soil, the rate is in a range of 0.2 - 20 μm/y, and greater than 20 μm/y for aggressive soils.

*Pipe ultimate strength* $\sigma_u$. The ultimate strength of the pipe material, $\sigma_u$, is found in the flexural yielding limit state function only. The variability of $\sigma_u$ is described using a normal distribution with a mean value and a standard deviation. Thus, to determine the effect of $\sigma_u$ on the life expectancy of a decommissioned pipeline, two separate cases were implemented, one comparing the life expectancy difference due to a change in the mean value of $\sigma_u$, the other case comparing the life expectancy difference due to a change in the standard deviation of $\sigma_u$. In the first case, the mean value of $\sigma_u$ was decreased from 760 MPa to 360 MPa (equivalent to the yield strength of X52 pipe steel). Fig. IV-9 illustrates the pipe life expectancy comparison between $\sigma_u = 360$ MPa and $\sigma_u = 720$ MPa. For both ultimate strength values, the standard deviation was kept the same at 25 MPa following a normal distribution. There is not a significant difference in the life expectancy when the mean value of $\sigma_u$ increase or decrease drastically. In the second case, the standard deviation of $\sigma_u$ was increased from 25 MPa to 50 MPa. The mean value was kept at 760 MPa. Fig. IV-10 illustrates the pipe life expectancy comparison between the two standard deviations. Once again, there is not a significant difference in the life expectancy when the standard deviation of $\sigma_u$ increase or decrease drastically. Thus, any change in the variability of $\sigma_u$ will not affect the life expectancy outcome of a decommissioned pipeline in a significant manner. It is important to note, however, the insignificant impact on the pipe life expectancy is due to a power law expression for the corroding thickness. A linear expression for the corroding thickness such as $w(t) = kt$ will have a greater impact on the pipe life expectancy if the mean value or standard deviation changes.
Fig. IV-9. Flexure yielding - Comparison of decommissioned pipeline life expectancy for two different mean values of pipe steel ultimate strength based on power law corrosion of Velazquez et al. (2009). $\sigma_u = 360$ and 760 MPa.

Fig. IV-10. Flexure yielding - Comparison of decommissioned pipeline life expectancy for two different standard deviations of pipe steel ultimate strength. $\sigma_u = 760$ MPa.

Pipe diameter $D$. The final parameter of interest in this series of parametric studies is the pipe diameter, $D$. The pipe diameter, $D$, is found in the flexural yielding limit state function only. A
different pipe diameter will also affect the burial depth, \( H \), as \( H = 1 + D/2 \). For comparison purposes, the pipe diameter \( D \) was decreased from 0.762 m to 0.3 m. Fig. IV-11 illustrates the life expectancy comparison for two different diameter sizes of a decommissioned pipeline. It is shown that the smaller diameter pipe (\( D = 0.3 \) m) has a slightly longer life expectancy than the larger diameter pipe. However, the increase in life expectancy due to a smaller pipe diameter is not as significant compared to varying the values of \( \kappa \) and \( \nu \) in the power law expression for the corroded pipe thickness.

![Graph](image)

**Fig. IV-11.** Flexure yielding - Comparison of decommissioned pipeline life expectancy for two different pipe diameters. \( \sigma_u = 760 \) MPa.

**Occurrence or “failure” probability.** The life expectancy of a decommissioned pipeline is dependent on the selected occurrence or “failure” probability. Fig. IV-12 shows the effect of \( \sigma_u \) on the life expectancy for varying uniform corrosion rates at a given constant probability of \( 10^{-4} \). Fig. IV-13 compares the life expectancies with varying occurrence or “failure” probabilities of \( 10^{-4} \) and \( 10^{-2} \) at a constant \( \sigma_u = 360 \) MPa. From Figs. IV-12 and IV-13, it is observed that the life expectancy is more sensitive to the occurrence or “failure” probabilities compared to \( \sigma_u \).
Fig. IV-12. Flexure yielding - Decay trend for decommissioned pipe life expectancy with an increasing mean corrosion rate at a critical occurrence probability, $P_f$, of $10^{-4}$. $\sigma_u = 360, 460$ and 760 MPa.
Fig. IV-13. Flexure yielding - Decay trend for decommissioned pipe life expectancy with an increasing mean corrosion rate at a critical occurrence probability, $P_f$, of $10^{-4}$ and $10^{-2}$. $\sigma_u = 360$ MPa.

5.5. Limitations and Future Considerations

The results presented in this study represent a very conservative estimate for the life expectancy of a decommissioned pipeline and likely not representative of the actual field conditions. This section addresses the limitations to the findings of this study and to propose future considerations in the next study.

5.5.1. Pitting corrosion versus uniform corrosion

In general, uniform corrosion rates are measured in laboratory based on measurement of mass loss in test coupons. Pitting corrosion rates are usually measured in sections along the pipeline in the field. Correlation between uniform and pitting corrosion rates must be developed (Fig. IV-14). Use of either rate in predicting the life expectancy of a decommissioned pipeline may or may not yield conservative results depending on the quantity of pits, the depth of each individual pit, and the spatial distribution of the pits along a pipe section. Future Monte-Carlo simulations need to reflect pitting corrosion through the conversion of a corroded uniform section into a corroded pitted section. In-line inspection is valuable in determination the spatial and temporal distribution of corrosion.
5.5.2. Extreme value distributions

While both limit state functions are recommended to be modified to account for pit corrosion, the modified limit state functions only describe one pit. Future Monte-Carlo simulations need to be completed for multiple pits within a pipeline section. For every event simulated with multiple pits, the maximum pit depth collected from each event can be used to create an extreme value distribution. The extreme value distribution of the highest pit depths will better represent the worst case scenario in finding the life expectancy of the decommissioned pipeline at a critical occurrence probability, $P_{f}$, of $10^{-4}$.

5.5.3. Inclusion of live loads into probabilistic analysis

The flexure yielding limit state function has only taken into consideration the critical bending stress due to soil overburden, which is the dead load only. A full limit state analysis should also incorporate the live load into the flexure yielding limit state function. The live loads that a pipeline crossing may encounter and should be considered in future analyses include heavy loads from the axles of farm tractors and trucks.

5.5.4. Pit corrosion multiplier $\beta$

The results of this study were conducted using a pit corrosion multiplier, $\beta$, of 1 in the flexure yielding limit state function. This is conservative since a $\beta$ of 1 represents uniform corrosion of the section. The value for $\beta$ should be examined further for pitting corrosion. Two cases to look at for future consideration include: (1) $\beta$ due to the initiation of $\sigma_u$ on the pipe surface (triangular stress distribution along the pipe wall thickness cross-section) and (2) $\beta$ at pipe full capacity (rectangular stress distribution along the pipe wall thickness cross-section, or plastic capacity). The
The $\beta$-factor is a function of pit depth and spatial distribution. Results based on a preliminary analysis are shown in Figs. IV-15 and IV-16 for different postulates.

(a) 

Fig. IV-15: Variation of $\beta$-factor with pit depth for initiation of ultimate strength, $\sigma_u$ at pipe wall outer edge with linear stress distribution across the section (a) $t/w_0 = 0.0001 - 0.3$, and (b) $t/w_0 = 0.3 - 0.75$ where $t$, $w_0$, and $d$ are uniform corrosion depth, initial pipe wall thickness, and pit depth, respectively.
Fig. IV-16. Variation of $\beta$-factor with pit depth for full mobilization of ultimate strength, $\sigma_u$ across the entire section (a) the assumed corroded section is made up of a constant $w_0$ and reduced width, and (b) the assumed corroded section is made up of a reduced $w$ and constant width where $t$, $w_0$, and $d$ are uniform corrosion depth, initial pipe wall thickness, and pit depth, respectively.
5.5.5. **Structural integrity**

Though the flexure yielding limit state function is used in the probability analysis in this study, it does not imply the pipe section would lose its structural integrity. The pipe section still remains structurally stable even yielding has initiated at the surface of the pipe well. Possible kinematically admissible collapse modes which result in soil subsidence must be studied.

5.5.6. **Selection of Occurrence of Probability**

The values for probability of failure are well defined in design of infrastructure (Table IV-4). A rational target total risk of failure was proposed [CIRIA, 1997]:

\[
P_f = \frac{10^{-4} K_s N_d}{N_r} \quad (IV-10)
\]

where \( P_f \) is the probability of failure due to any cause in design life \( N_d \) years, \( N_r \) is the number of people at risk in the event of failure, and \( K_s \) is 0.005 (places of public assembly, dams), 0.05 (residential and commercial buildings), 0.5 (bridges), and 5 (towers, masts of offshore structures), respectively. For decommissioned pipeline, it is a new research subject which requires further studies.

**Table IV-4. Tentative minimum acceptable probability of failure based on marginal lifesaving costs principle [ISO, 2015]**.

<table>
<thead>
<tr>
<th>Relative marginal lifesaving costs</th>
<th>Acceptable ( P_f )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Large (I)</td>
<td>( 10^{-3} )</td>
</tr>
<tr>
<td>Medium (II)</td>
<td>( 10^{-4} )</td>
</tr>
<tr>
<td>Small (III)</td>
<td>( 10^{-5} )</td>
</tr>
</tbody>
</table>

5.6. **Summary**

There are two methods in dealing with decommissioned pipelines. One method is the excavate along the right-of-way of the decommissioned line, pull out the pipe, and refill the backfill.
However, this method is not economical. The other method is the leave the decommissioned line in the ground. However, pipe corrosion may cause (i) soil subsidence due to soil migration through corroded pits, and (ii) potential collapse with imposed soil and live loads. Two limit state functions of pitting corrosion and flexure yielding were developed to estimate the life expectancies, respectively. A Monte-Carlo simulation of 100,000 events was completed for each limit state function at a specific time. For pitting corrosion limit state function based of the power law corrosion [Velazquez et al., 2009], the estimated life expectancy is 63 years for a probability of occurrence of pitting corrosion of $10^{-4}$ (Fig. IV-4). For higher occurrence probabilities of $10^{-3}$ and $10^{-2}$, the estimated life expectancies are increased to 81, and 102 years, respectively. For flexure yielding limit state function with the structural reliability criterion of a critical occurrence probability of $10^{-4}$, the estimated life expectancy of the decommissioned pipeline is about 1,600 and 170 years for uniform corrosion rates of 0.2 and 20 µm/y, respectively (Fig. IV-12). For more aggressive corrosion rates, the life expectancy would be reduced. Though uncertainties are embedded in the probability analyses, the behavior of a decommissioned pipeline is governed by long-term corrosion progression.
6. References


