GEO
ST. JOHN'S
2019
UNDER LAND & SEA

Framework for Assessing Integrity of Natural Gas Distribution Pipes in Landslide Areas

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ABSTRACT
A framework was developed to assess and manage the gas distribution system in areas impacted by slow moving landslides recognizing that ruptures or leaks in the system could pose significant damage to the public, properties, environment and reputation. This paper discusses the analytical approaches developed to assess the hazard (i.e., likelihood of failure) and the risk management approach implemented through monitoring and inspection programs. The analytical methods have been developed to correlate the measured ground displacement to pipe strain (or stress) by considering the pipe-soil interaction. Once the ground displacement is known, a screening-level analysis is performed to identify the pipe segments with high likelihood of failure. The threshold ground displacement levels corresponding to different hazard ratings have been developed using these analytical empirical fragility models for different pipe sizes by considering typical soil and burial conditions. Once identified, the higher risk areas are analyzed using site-specific conditions. Although limited in size, the existing leak database indicate this approach accurately identifies the critical pipe components and time when remediation measures are required. The paper discusses the implementation of this approach in three landslide areas in British Columbia.

RÉSUMÉ
Un cadre a été mis au point pour évaluer et gérer le système de distribution de gaz dans les zones touchées par des glissements de terrain lents, reconnaissant que des ruptures ou des fuites dans le système pourraient causer des dommages importants au public, aux biens, à l'environnement et à la réputation. Ce document traite des approches analytiques développées pour évaluer le danger (c'est-à-dire la probabilité de défaillance) et de la méthode de gestion des risques mise en œuvre par le biais de programmes de surveillance et d'inspection. Les méthodes analytiques ont été développées pour corréler le déplacement du sol mesuré à la déformation (ou contrainte) d'un tuyau en prenant en compte l'interaction tuyau-sol. Une fois que le déplacement du sol est connu, une analyse préliminaire est effectuée pour identifier les segments de conduite présentant un risque élevé de défaillance. Les seuils de déplacement du sol correspondant à différentes cotes de risque ont été élaborés à l'aide de ces modèles de fragilité empiriques analytiques pour différentes tailles de conduites, en tenant compte des conditions types de sol et d'enfouissement. Une fois identifiés, les zones à haut risque sont analysées à l'aide de conditions spécifiques au site. Bien que de taille limitée, la base de données de fuites existante indique que cette approche identifie avec précision les composants critiques de la conduite et le moment où des mesures de correction sont nécessaires. Le document traite de la mise en œuvre de cette approche dans trois zones de glissements de terrain en Colombie-Britannique.

1 INTRODUCTION

The integrity of the natural gas distribution pipe systems in slow moving landslide areas is a key concern to FortisBC Inc. (FortisBC) because of the potential social, economic and environmental impact that may arise from a leak in the pipeline. Evaluating the integrity of these distribution pipe networks is a difficult undertaking due to its intricacy and potentially large area impacted by the movement. Even if the magnitude of ground movement is known through a comprehensive monitoring program, an accurate estimation of the likelihood of pipe failure has become a difficult undertaking. To the best of authors' knowledge, there are no reliable methods to assess the integrity of distribution networks in landslide areas, although such methods have been developed for larger diameter transmission pipes. The conventional pipe-soil interaction techniques developed for transmission pipes are not directly applicable to a distribution pipe network. Often, pipe integrity assessments in distribution systems are difficult due to their smaller pipe size, intricate configurations and components, complex pipe material characteristics associated with plastic pipes and lack of leaks to calibrate or validate analytical methods. Over the last decade, with the aid of academic and consulting industry, a systematic approach has been developed to assess and manage the gas distribution system in areas impacted by slow moving landslides. The hazard (i.e., likelihood of pipe leak) was estimated using experimental, analytical and numerical methods and the risk management approach implemented through monitoring and inspection programs. Some of the existing analytical methods were improved. This paper provides an update of the framework and discusses the implementation of this to three landslides in British Columbia (BC).

At present, FortisBC have been monitoring a couple of large-scale slow-moving landslides in BC, including the slide in West Quesnel, Marble Hill Subdivision slide in

Chilliwack, and Hodgson Road slide in Williams Lake. Locations of these landslides are shown in Figure 1. Apart from the Hodgson Road slide, the gas distribution networks in these areas consist of medium density polyethylene (MDPE) pipes. Some steel pipe sections are encountered within the Hodgson Road slide, although they are gradually being replaced by MDPE pipes. The pipe diameters in gas mains range from 42 to 114mm while most service pipes consist of pipes with diameters ranging from 15 to 25mm.

Figure 1. Three landslide locations in British Columbia

The deep-seated landslide in West Quesnel was first conclusively identified in an investigation commissioned by the City in 2000-2002. In general, the ground movements have been occurring at a rate of 20 to 70mm per year, and has impacted approximately 940 parcels of land, 750 homes, one elementary school and several businesses. Between 2003 and 2011, the City commissioned additional geotechnical and hydrogeology studies to outline the boundaries of the affected area and to provide recommendations to mitigate the impact. Based on these studies, a dewatering program was initiated in 2013 which appeared to have reduced the rate of ground movement to less than 20mm per year. Further details of the slide can be found in series of reports prepared by Wood Consultants Ltd (formerly AMEC Foster Wheeler). We are currently monitoring the movements in every four months using over 80 survey hubs, while real-time monitoring data is also collected by Wood Consultants Ltd as a part of the dewatering program. Real-time monitoring has revealed movement rates of about 0.25 to 4mm/day occurring in relatively shorter periods. The rate of movement is an important consideration since the stress-strain behavior of MDPE pipe material is strain-rate dependent. In 2001, FortisBC replaced the steel gas distribution piping system with MDPE in an attempt to improve the resiliency of the gas pipe network. Since then, only few leaks have been reported in the pipe network.

The Marble Hill subdivision is situated on the north slopes of the mountain ridge that separates Chilliwack River Valley from the flat agricultural land of the Fraser River Valley. The ground slopes toward the north with gradients more than 30% in certain areas. The development of the Marble Hill subdivision began in the early 1990s and the most pipes were installed between 1993 and 1994. MDPE pipes have been used in this area since the development of this subdivision. The first evidence of ground movement came to light in the late 1990s, when cracks in houses and water main breaks were reported. Following these incidents, the City of Chilliwack enforced a “Panorama No-Built” zone in 2002. Over 100 ground survey hubs were installed by the City of Chilliwack in 2004 and 2009. The gas mains were replaced between 2015 and 2017 because of concerns related to the ground movement magnitudes measured since the initial installation and increasing number of leaks. In 2016, additional 43 survey hubs were installed, of which 34 were located on the pipe itself and the remaining nine survey hubs were located on the ground surface. During the recent pipe replacement, eight strain gauge arrays were installed on pipes to obtain direct pipe strain measurements. The annual ground displacement rate ranges from about 12 to 120 mm/year with the maximum measured at a survey hub located at the upslope of the slide and away from the subdivision. Since the monitoring commenced in 2004, the ground movement rates remained almost constant. The details of the slide can be found in series of reports prepared by Klohn Crippen Berger for the City of Chilliwack.

The Hodgson Road slide is located west of Highway 20 on the south side of Williams Lake, BC. Unlike the other two slides, the Hodgson slide has been moving episodically with greater movements reported during 1993-1997 and 2012-2015 periods. During these periods, the measured ground movement was rapid, for example, movement rates in the order of 0.5 to 1.0mm/day was measured in 1996 (Golder Associates, 1997), and annual displacements reaching as much as 100 to 150 mm/year (Evergreen Geotechnical Inc, 2015). In 2015, Golder Associates estimated the total displacement to range from 300 to 400m since the initial survey was conducted in 1993. However, the exact magnitude of the ground displacement is uncertain as the survey was discontinued after the initial monitoring. Further details related to the slide can be found in series of reports prepared by Golder Associates and Evergreen Geotechnical Inc. Since 2015, slide has been monitored using 22 survey hubs and additional 11 survey hubs will be installed in 2019. As leaks have been noticed in some steel pipe sections, two steel pipe sections were replaced with MDPE pipes between 2015 and 2016.

Although the slip surfaces are deep in all three areas, the slide geometries appear to be complex with several smaller and shallower slides occurring within the main slide mass and causing localized damages. In general, the most structural damages, broken water and sewer lines and deformed roadways have been reported near the slide boundaries where differential movements between stable and unstable soil masses occur.

Apart from the West Quesnel slide, no active remediation/mitigation strategies have been implemented by the stakeholders for the other two slides. While respective Cities and other stakeholders are responsible for managing the landslide and its impact on residents and its infrastructure, FortisBC efforts are limited to managing the integrity of the gas pipelines.

The main challenge is to estimate the current condition of the pipe based on the measured ground displacement such that effective and timely remediation measures can be implemented. First and foremost, there are considerable difficulties in conducting pipe-soil interaction assessments for complex pipe networks that spans over a large geographical area. For example, within the 2km^2 area impacted by the West Quesnel slide, over 17 km of buried pipes are included. Existing analytical methods have been developed for large-diameter steel transmission pipelines and there is considerable uncertainty when attempting to adopt these methods to analyze a complex distribution system that consists of smaller diameter pipes. Typically, pipe-soil interaction analysis becomes somewhat straightforward if the impacted pipe length is limited to a relatively smaller length such that it can be modeled relatively easily using numerical or analytical tools (e.g., pipe crossing a river or an active fault). Given the large geographical extent and smaller diameter pipes, many survey hubs are required to map the slide geometry and displacement variations within the slide mass. Unlike for a transmission pipe, analysis becomes complex due to frequent pipe connections in the distribution pipe network. Most failures occur at these pipe connections compared to the pipe itself. Further, Weerasekara (2011) demonstrated the importance of soil dilation on smaller diameter pipes when ground movement occurs along the pipe axis. The impact of soil dilation becomes negligible as the pipe diameter increases. For MDPE pipes, pipe–soil interaction analysis becomes further complicated due to temperature and strain rate dependency of the pipe material stiffness. Apart from the above difficulties, estimating the strain in a service conduit that has a diameter of 25 mm or less is beyond the analytical capabilities even if all other concerns are addressed. Ironically, lack of historical leaks in MDPE pipes cause challenges in attempting to confirm the accuracy of the prediction methods or to develop empirical fragility curves that link ground displacements to pipe failures.

Note that in this paper, gas leaks and ruptures are collective called as “failures” due to difficulties in predicting the failure mode in advance. A “leak” is a pipeline failure where a pipeline is losing its product but might continue to operate until the leak is detected, whereas a “rupture” is a pipeline failure where a pipeline cannot continue to operate. As expected, rupture may lead to a catastrophic consequence than a leak.

2 FRAMEWORK FOR PIPE INTEGRITY ASSESSMENT

In contrast to analyses performed for larger diameter pipes, sole reliance on analytical and numerical modeling is not sensible due to the challenges highlighted earlier. Thus, additional field monitoring and instrumentation are required to supplement the analytical/numerical model predictions. With this background, the integrity of the distribution pipe system is assessed using the following tools to provide a holistic interpretation of the pipe condition instead of relying on a single method:

1. Soil–pipe interaction analysis using analytical and numerical models,
2. Pipe integrity dig programs
3. Periodic inspection of above-ground pipe components such as riser pipes, gas meters and building structure
4. Strain gauging of live pipes (only in Marble Hill slide)
5. Leak surveys

Due to space limitations, this paper predominantly discusses the analytical tools developed to conduct pipe-soil interaction analyses.

3 PIPE-SOIL INTERACTION ANALYSIS

The main objective of the analytical methods discussed in this paper is to quantify the pipe strain/stress based on the measured ground displacement, therefore considered as a key component of the pipe integrity assessment. FortisBC’s research collaboration with the University of British Columbia (UBC) has led to the development of an improved analytical model to estimate the pipe strains for MDPE pipes. Arising from this research undertaking, Weerasekara (2011) proposed two analytical models for estimating the pipe strain caused by ground displacements along the pipe axis (i.e., axial) and perpendicular to the pipe axis (i.e., lateral). Recently, the model was extended to steel pipes in which the solutions have become simpler due to constant pipe material stiffness. A brief description of these models is given below.

3.1 Ground Movements along Pipe Axis (Axial)

The analytical model is based on an improved interface friction model that considers the increased interface friction due to constrained soil dilation around the pipe. The increased friction from soil dilation and subsequent degradation of friction with increasing displacement is modeled using a simplified model proposed by Weerasekara (2011), and schematically shown in Figure 2 below.

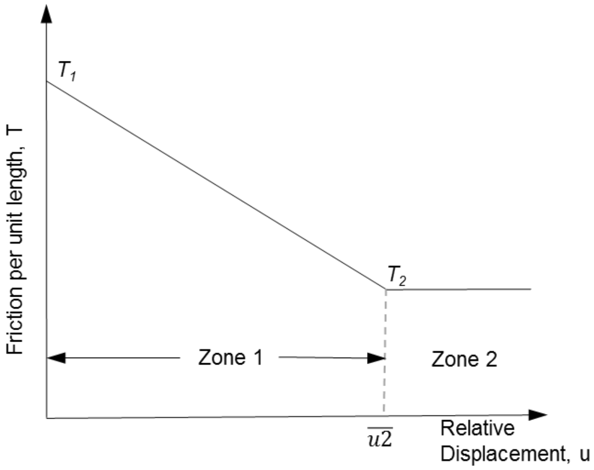


Figure 2: Interface friction model.

In this model, the peak frictional resistance per unit length of pipe (*T_1*) can be expressed as follows:

$$T_1 = (\Delta\sigma_d + \sigma')\pi D \tan\delta \qquad [1]$$

Where, $\sigma' = 0.5(1 + K_0)\gamma H$ and $\Delta\sigma_d$ is the net normal pressure increase on the pipe due to soil dilation. Also, *D* is the pipe diameter, *H* is the burial depth measured from ground surface to pipe springline, γ is the soil unit weight, *K_0* is the lateral earth pressure coefficient under at-rest conditions, and δ is the interface friction angle between pipe and soil. At large displacements, $\Delta\sigma_d$ approaches zero and the unit interface friction becomes equal to T_2 as shown in Figure 2.

For a cylindrical object, $\Delta\sigma_d$ can be approximately estimated from the elastic cavity expansion theory (Boulon and Foray, 1986, Johnston et al., 1987). Luo et al. (2000) proposed an approach in which a saw–tooth type model was used to represent the soil dilation, from which $\Delta\sigma_d$ is expressed as a function of shear modulus of the soil (*G*) and radial displacement associated with the peak soil dilation (Δt_d). The potential for soil dilation further depends on the mean effective stress (σ') around the shear zone; as such, the dilation will be suppressed when σ' increases around the pipe. To account for this effect, Luo et al. (2000) proposed to use Bolton (1986) empirical relationship to replace the maximum dilation angle in the cavity expansion equation. As a result, the relative density (I_D) and empirical parameters in Bolton's equation (*Q* and *R*) are introduced into the equation, and results in the following equation for $\Delta\sigma_d$. Further details related to the derivation can be found in Weerasekara (2011).

$$\Delta\sigma_d = \frac{2G\Delta t_d}{D}\tan(3I_D(Q - \ln\sigma') - 3R) \qquad [2]$$

The displacement at which the dilation become negligible (i.e., $\overline{u_2}$) can be obtained from the experimental results published by Scarpillai and Wood (1982), Stone and Muir Wood (1992), Vardoulakis et al (1981) etc. This value generally observed to range from about 100 to 176 times the mean particle size (*d_{50}*). Similarly, Δt_d can also be obtained from experimental results, which ranges from about 10 to 20 times d_{50} (Roscoe 1970, Scarpillai and Wood 1982, Luo, et al 2000). Further details related to estimation of *T_1, T_2* and $\overline{u_2}$ values are given in Weerasekara (2011).

For MDPE pipes, the above interface friction model was combined with a nonlinear and strain rate dependent pipe stiffness model to derive a second-order differential equation by considering the equilibrium of a pipe element. This model forms a framework to determine force (or pipe stress), strain and mobilized length along the pipe for a measured ground displacement. Large-scale pipe pullout tests were conducted to verify the results obtained from the analytical model (Wijewickreme et al. 2015). The analytical model predictions were in good agreements with test results conducted at different soil/burial conditions, displacement rates and pipe diameters. The details of the model validation are given Wijewickreme et al. (2015).

For steel pipes, a similar closed-form solution can be developed by considering a constant material stiffness as follows:

$$\frac{d}{dl}\left[EA\left(\frac{du}{dl}\right)\right] = \mathrm{T} \qquad [3]$$

E is the pipe modulus, *A* is pipe cross sectional area, *u* is the relative displacement and *l* is the mobilized length along the pipe. To further simplify the solution, *T* can be written in the following form

$$T = EA\,(\kappa - \lambda u) \qquad [4]$$

Where, $\lambda = \frac{(T_1 - T_2)}{EA\overline{u_2}}$ and $\kappa = \frac{T_1}{EA}$

The relative displacement and strain in the reinforcement are obtained by integrating Equation 4 as follows:

$$\mathrm{u} = \mathrm{C}_1 Cos\sqrt{\lambda}l + \mathrm{C'}_1 Sin\sqrt{\lambda}l + \kappa/\lambda \qquad [5]$$

$$\varepsilon = \sqrt{\lambda}\mathrm{C}_1 Cos\sqrt{\lambda}l + \sqrt{\lambda}\mathrm{C'}_1 Sin\sqrt{\lambda}l \qquad [6]$$

where *C_1* and *C'_1* are constants, and $\sqrt{\lambda}l < \frac{\pi}{2}$. The two unknowns in Equations 5 and 6 can be determined using the following boundary conditions: At *l* = 0, *u* = 0 and ε = 0 which results in $C_1 = -\kappa/\lambda$ and $C'_1 = 0$. Substituting these values in Equations 5 and 6, the following governing equations are obtained.

$$u = \left(\frac{\kappa}{\lambda}\right)\left(1 - \cos\sqrt{\lambda}l\right) \qquad [7]$$

$$\varepsilon = \left(\frac{\kappa}{\sqrt{\lambda}}\right)\left(\sin\sqrt{\lambda}l\right) \qquad [8]$$

If the displacement is known, the mobilized frictional length along the pipe can be obtained by rearranging Equation 7 as follows:

$$l = \left(\frac{1}{\sqrt{\lambda}}\right)\cos^{-1}\left(1 - \frac{u\lambda}{\kappa}\right) \qquad [9]$$

Knowing the strain, axial force developed in the pipe at any given location can be obtained as follows:

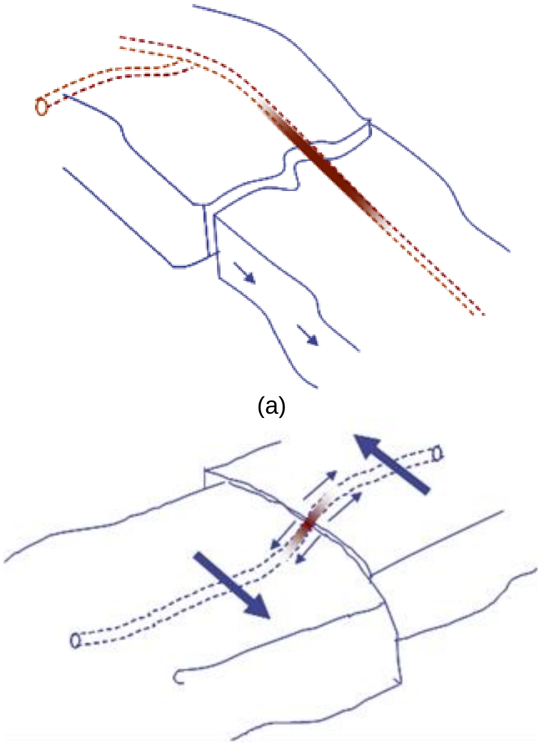
$$P = E\,A\,\varepsilon \qquad [10]$$

Using a similar approach, the governing equations for Zone 2 can be obtained, although it is not discussed for brevity.

3.2 Ground Movements Perpendicular to Pipe Axis (Lateral)

For lateral loading, a close-form solution can be developed based on the beams-on-elastic foundation theory (Hetenyi, 1946). A bilinear lateral soil spring was considered as recommended by Honegger and Nyman (2004). The model derivation is given in Weerasekara (2011), therefore not repeated herein. For simplicity, a constant pipe stiffness was considered, and the resulting closed-form solution was implemented in a spreadsheet.

On average, the distance between two survey hubs is about 150 m even for the West Quesnel slide where more than 80 survey hubs are installed. Analytical and experimental results indicate that for a small-diameter MDPE pipe, the impacted length from an abrupt ground movement could be as less than 10 m depending on the pipe size and burial condition (Weerasekara, 2011). In the absence of survey hubs to estimate the differential movement at such short distances, a prudent approach would be to assume an abrupt differential movement at the midpoint of two survey hubs as schematically shown in Figure 2. This is a conservative assumption, but considered reasonable in the absence of evidence to show more evenly distributed differential movements between the hubs.


(a)

(b)

Figure 3. Schematic representation of abrupt (a) axial and (b) lateral ground movement.

3.3 Likelihood of Failure

Using the analytical models discussed above, strains in the gas mains can be estimated. However, it is considered not practical to undertake such analyses for service pipes of about 25 mm or smaller due to the uncertainties related to the installation and behavior of these conduits. Once the analytical models are in place, an initial screening was performed by dividing the distribution network into smaller segments based on the pipe material type, diameter, pipe installation date and loading direction. For example, the 17 km long pipe network in West Quesnel was discretized into 63 pipe sections. The main objective of this analysis is to identify the pipe sections with higher likelihood of failure. For this purpose, the likelihood of failure, is categorized as "low", "medium" and "high". For reasons given later in the paper, both displacement and strain based threshold limits have been considered as shown in Table 1.

Table 1. Threshold strain and displacement selected for different hazard ratings

Likelihood of Failure	Threshold Strain and Displacement	
	MDPE	Steel
Low (less than)	min(4% strain, 100 mm displacement)	min(0.15%, 100 mm displacement)
Medium (between)	min(4 to 8% strain, 100 to 225 mm displacement)	min(0.15 to 0.2% strain, 100 to 300 mm displacement)
High (greater than)	min(8% strain, 225 mm displacement)	min(0.2% strain, 300 mm displacement)

At present, FortisBC adopts an "allowable" strain limit of 8% for MDPE pipes for axial (compressive and tensile) and bending modes of failures. Using this as the reference, any pipe section experiencing greater than 8% axial strain is classified as "high" likelihood of failure, while pipe sections with strains less than 4% are classified as "low" likelihood of leaking. Using the strain-based analytical model discussed previously for MDPE pipes, high likelihood of failure will be identified if the relative axial ground displacement exceeds 120, 195 and 350 mm for 42, 60 and 114 mm pipes respectively. For MDPE pipes subject lateral soil loading, the ground displacement corresponding to 8% bending strain is 320, 600 and 1500 mm for the 42, 60 and 114 mm pipes respectively.

Above strain estimates are only applicable for the gas main and ignores the stress concentrations near tapping tee connections. As demonstrated in the analytical models, a larger diameter MDPE pipe can accommodate relatively large ground displacements, although leaks may occur at a smaller displacement level at or near a tapping tee connection due to the localized stresses.

Although limited in size, review of the existing leak database from the Marble Hill slide indicate that most leaks occur when the differential displacement exceeds about 225 mm (9"). The leak database is limited in size and consists of 60 mm and 42 mm pipes (114 mm diameter pipes are not encountered in this pipe network). At locations where leaks have been observed, the ground displacement occurred mostly along the pipe axis. The displacements associated with these leaks are plotted on Figure 4 and the displacement thresholds corresponding to strain-based ratings adopted for MDPE pipes are shown in the background for comparison. While the analytical model can estimate the strain in the pipe itself, the localized strains and its potential to cause leaks near the tapping tees could only be accounted using the above fragility relationship that links the displacement to pipe leaks. If tapping tees are not encountered, the analytical model alone is sufficient to estimate the likelihood of failure. Until additional leak information become available, it is considered reasonable to adopt a displacement threshold of 225 mm as the lower limit of high likelihood of failure rating unless a lower displacement threshold is predicted using the analytical model for a threshold pipe strain of 8%. At present, this threshold displacement is considered applicable to all three pipe sizes and loading modes, although there are no historical leaks to validate this observation for 114 mm diameter pipes or larger.

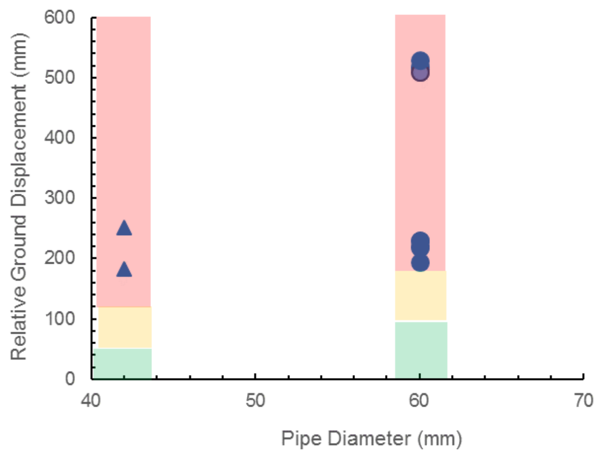


Figure 4. Displacements corresponding to leaks shown as blue triangles and circles (Marble Hill slide). The likelihood of failure ratings derived from analytical models for MDPE pipes are shown in the background for comparison (i.e., red: high, yellow: medium, and green: low likelihood of failure).

For steel pipes, yield strain was selected as the threshold for the “high” likelihood of failure ranking, although it is recognized that these pipes could sustain significantly larger strains before pressure integrity is compromised. Similar to MDPE pipes, this threshold strain limit was considered applicable to all failure modes (i.e., tension, compression or bending) despite different threshold limits may correspond to different modes of failure. In areas where ground displacements have been reported, leaks at the dressers were found to be the most common form of failure, thus considered as an excellent indicator of the ground movement hazard. The dressers are introduced into the system with the objective of protecting the gas main and riser pipes from excessive pulling forces, and typically located at or near the property line. Furthermore, dressers prevent significant strains being developed in the service pipe due to the slippage at the dresser. For these reasons, strain based thresholds are generally poor indicators of the failure likelihood, and it is considered more appropriate to use an empirical displacement threshold for steel pipes when dressers are encountered. As stated earlier, except for a limited number of steel pipes within the Hodgson slide, no steel pipes are encountered in these three landslide areas. The leak information collected from steel pipes located in other areas in BC reveals that dresser failures are more frequent when the ground displacement exceeds about 300 mm. This is an approximate estimate which was established from observed differential settlements at buildings and road pavements. Despite the large leak database, no direct ground survey measurements are available in these areas to develop a more precise relationship between dresser failures and ground movement. Similar to MDPE pipes, a strain based threshold limit is only applicable if dressers are not present (see Table 1). It should also be noted that dressers are not used in certain areas in BC.

Using the strain and displacement based thresholds discussed above, a single displacement limit can be established for pipe networks consisting dressers and tapping tee connections. These displacement thresholds are summarized in Table 2 for different hazard ratings, pipe sizes and loading modes. Figures 5 through 8 also show these differential ground displacement thresholds, along with pipe strain estimates from analytical models. Again, authors would like to highlight that analytical model predictions ignore the presence of tapping tees and dressers, while the hazard ratings shown in the background incorporates the impact from these pipe components. To develop these plots, typical burial and soil conditions were considered. For example, pipe wall thickness was calculated using a Standard Dimension Ratio (i.e., diameter to pipe wall thickness ratio) of 10 for MDPE pipes, and schedule 40 pipes for steel pipes. Furthermore, a pipe burial depth of 0.6 m and a soil friction angle of 36 degrees were considered. More detailed analyses using site-specific information could be performed once the critical pipe segments are identified from this screening analysis. If input parameters and ground movement data are sparse, a detailed analysis is not expected to yield a greater accuracy.

Table 2: Displacement thresholds derived for different hazard ratings for a pipe network consisting tapping tee connections and dressers

Pipe Diameter	MDPE Pipes			
	Axial		Transverse	
(mm)	Low	High	Low	High
42	<50	>120	<100	>225
60	<95	>195	<150	>225
114	<165	>225	<150	>225
Pipe Diameter	**Steel Pipes**			
	Axial		Transverse	
(mm)	Low	High	Low	High
42	<50	>115	<30	>85
60	<75	>180	<40	>120
114	<185	>300	<180	>300

As indicated in Figures 5 through 8, MDPE pipe itself can typically tolerate a larger displacement in the lateral direction than in the axial direction. While a larger diameter pipe can accommodate a greater displacement, the maximum allowable displacement will largely depend on whether a tapping tee is present or not. Due to the subjectivity of the threshold strain levels selected for the two pipe material types (e.g., 8% for MDPE versus 0.2% for steel as the threshold for high hazard rating), direct comparisons between steel and MDPE pipes may not be appropriate. However, if this subjectivity is ignored, these figures indicate that MDPE pipes can accommodate a much larger ground displacement than steel pipes if the ground displacement occurs perpendicular to the pipe axis, while no significant differences between the two pipe material types are noticed if the pipe loading occurs along the pipe axis.

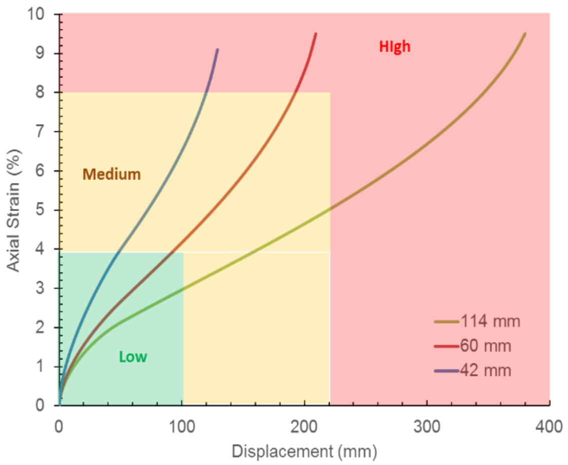


Figure 5. Maximum axial strains estimated from the analytical models and hazard rating for MDPE pipes (ground movement along the pipe axis).

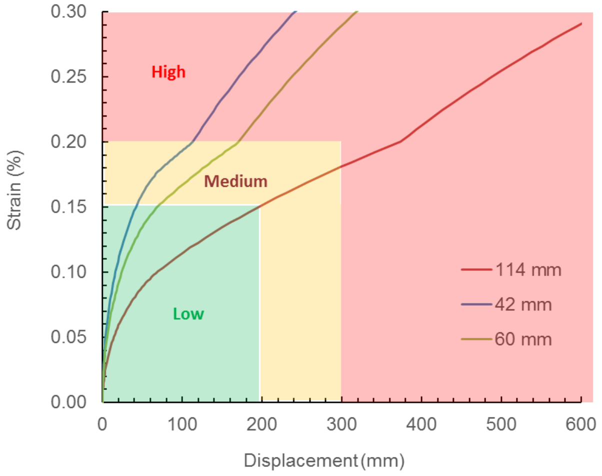


Figure 6: Maximum axial strains estimated from the analytical models and hazard rating for steel pipes (ground movement along the pipe axis).

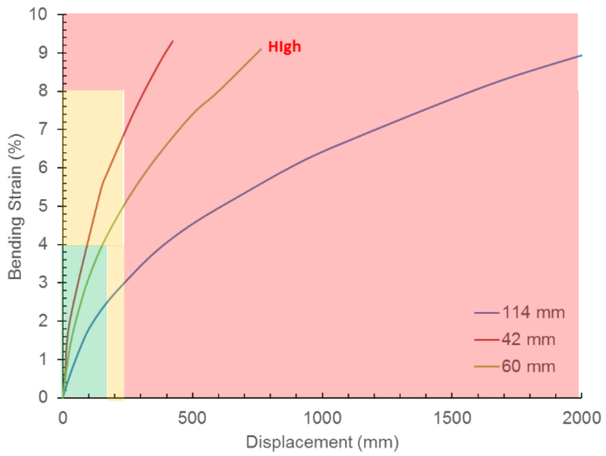


Figure 7: Maximum bending strains estimated from the analytical models and hazard rating for MDPE pipes (ground movement perpendicular to the pipe axis).

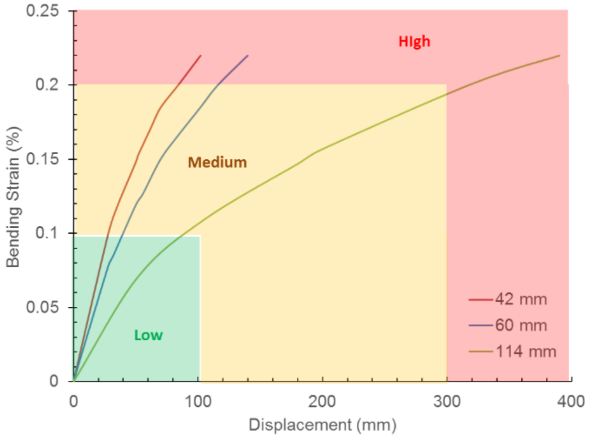


Figure 8: Maximum bending strains estimated from the analytical models and hazard rating for steel pipes (ground movement perpendicular to the pipe axis).

4 RESULTS AND DISCUSSION

The screening analysis demonstrated that most critical pipe sections are identified at the boundaries of the landslide mass, which is consistent with the general expectation. Once these critical sections have been identified, additional survey hubs can be placed along these pipe sections to obtain a more accurate estimate of the differential ground displacement. For example, Figure 9 shows such critical pipe section along Abbott Drive in the West Quesnel slide where nine additional survey hubs were installed in 2011. The survey hub measurements indicated that differential displacements are smaller and more gradual compared to the abrupt ground displacement assumed for the screening-level analysis using two survey hubs located far apart. The subsequent site-specific analyses and dig programs indicated no immediate threat to their integrity. Nonetheless, periodic leak surveys, ground movement monitoring and annual dig programs are continued in these areas.

As of 2018, two steel pipe sections located within the head scarp and toe of the Hodgson slide mass were rated as high likelihood of failure. In 2016 and 2015, two steel pipe sections located near the head scarp and toe of the slide were replaced after a number of leaks were reported in these pipes. It is interesting to note that, analytical approach in this paper also identify them as higher likelihood of failure prior to their replacement in 2015 and 2016, indirectly confirming the appropriateness of the model as an indicator of the likelihood of pipe failure. The replaced pipe segments were located adjacent to the two pipe segments assigned with a high likelihood of failure rating in 2018. Despite these general agreements, there is a greater uncertainty in the predictions for the Hodgson slide due to the limited number of survey hubs and lack of ground displacement data prior to 2015. Although the proposed framework appears to identify the critical pipe sections, integrity dig programs, leak surveys, monitoring, numerical and experimental studies are also required to supplement the conclusions of the model predictions. Until further field validation studies are conducted, the strains estimated using these analytical models should be treated only as an index of the likelihood of pipe failure instead of the actual strain experienced by the pipe itself.

Figure 9. Additional survey hub installed along Abbott Drive (the West Quesnel slide) to improve the accuracy of the differential displacement measurements.

As indicated earlier, there is greater uncertainty associated with strain estimations in MDPE pipes due their complex material behavior. MDPE pipes are viscoelastic and their stress-strain response is strain-rate dependent (i.e., estimated pipe strain and stress will depend on the ground displacement rate). Limited experimental results have indicated potential for “strain redistribution” with time, as such the maximum strain may not necessarily equal to strain estimated using the total differential movement measured since the pipe installation. For example, field pipe pullout tests performed by Weerasekara (2011) indicated about 30% drop in the maximum strain within a period of ten days, while the mobilized frictional length increased by about 30% during this period. The closed-form analytical models discussed in this paper are not capable of accounting for such complex behavior. Further testing is currently underway at the Memorial University of Newfoundland (MUN) to investigate the impact of strain redistribution.

The most serious limitation of these analytical models is the inability to estimate localized strains in the gas pipe due to tapping tee connections. As indicated earlier, most leaks appeared to have occurred at or near the tapping tee. A semi-empirical method was proposed by Weerasekara (2007) to estimate the anchoring resistance and its impact on the gas main based on limited experimental results. Additional pipe pullout tests are currently undertaken at MUN to quantify these localized strains near the tapping tee and their potential impact to the integrity of the pipe. Until more accurate prediction models are developed, semi-empirical method proposed in this paper could be adopted (Table 1).

The critical pipe sections identified from the pipe-soil interaction analysis were exposed and examined during “pipe integrity dig programs”. There are several limitations in attempting to use dig program observations to validate the analytical model predictions. For example, when a pipe section is exposed, the locked-in pipe stresses are almost immediately released, making it impossible to predict the condition of the pipe in its original burial condition. Furthermore, visual inspections would not be sufficient to estimate the pipe strain. Despite this limitation, deformations of the pipes near tapping tees have confirmed that certain pipe sections have been subject to ground displacement. Once exposed, some pipes are cut to measure the extension or compression of the pipe which can be considered as an indicator of the built-in pipe stress. However, there are no direct methods to quantify the pipe strain using the measured extension or compression.

To obtain direct pipe strain readings, strain gauges have been mounted on live gas pipes recently installed in the Marble Hill slide area. The measured strains are not significant as the pipes have been in place for only about 2 to 3 years. In addition to the above, periodic leak surveys and inspections are undertaken in these areas. As highlighted earlier, sole reliance on analytical methods is not recommended, while a holistic interpretation of the pipe condition can be made only if additional monitoring and instrumentations are undertaken to supplement the analytical findings.

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