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# Semi-empirical Framework to Predict Pipeline Failure Rates from Permanent Ground Displacement for a Regional Scale Seismic Vulnerability Study

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## ABSTRACT

In a regional-scale seismic vulnerability study, pipe damage rates are typically estimated using empirical relationships such as those published in American Lifeline Alliance guidelines. These existing empirical relationships generally account only for a limited number of parameters, such as the magnitude of permanent ground displacement, and fail to recognize other critical factors impacting the pipeline response, including the pipe diameter, pipe wall thickness, material strength, burial depth, corrosion, soil parameters and the direction of ground movement relative to the pipe axis. Furthermore, most empirical fragility relationships have been developed based on the performance of small diameter (less than 300 mm) brittle pipes; as such, there is considerable uncertainty in applying those relationships to estimate damage rates of large diameter ductile pipes. In this study, an attempt was made to estimate the relative vulnerability of larger diameter watermains using a pipe-soil interaction (PSI) analysis framework. The PSI analyses typically provide an estimate of pipe strain, which is not directly linked to damage rate. The purpose of calculating the damage rate, often expressed as the number of pipe repairs per unit length of pipe, is to facilitate post-earthquake recovery planning. A semi-analytical formulation is developed to estimate the damage rates from PSI analysis by incorporating factors such as the pipe length between two joints, probability of failure of the pipe for a given strain range, and corresponding pipe length and factors to account for corrosion, built-in pipe stresses from soil settlement, age, and welding type. In this method, the pipe strains and corresponding pipe lengths are estimated from the PSI analysis. One of the key advantages of this approach is the ability to incorporate the key influencing parameters in a fundament manner. The paper discusses the details of the proposed approach and key observations when the approach is utilized to estimate the damage rates for different pipe diameters and ground displacement magnitudes.

Keywords: Pipelines, regional-scale; fragility curves; pipe-soil interaction; empirical; welded; steel; watermain

## INTRODUCTION

Historical evidence shows permanent ground displacements (PGDs) as the main source of damage to buried pipelines during earthquakes. PGD can manifest in several forms including fault movements, landslides, lateral spreading and settlement. Liquefaction can occur in loose, saturated and relatively clean cohesionless soils where strong ground shaking causes an increase in porewater pressures within the soil deposit. The dissipation of excess porewater pressures will cause soils to settle and the amount of settlement depends on the ground motion intensity, relative density of the soil, and thickness of the liquefied deposit, among other factors. The combination of sloping topography and strength loss associated with seismic shaking can cause ground displacement in horizontal and vertical directions. Even in the absence of soil liquefaction, slope instability (landslide) may occur in relatively steep slope areas due to the inertial driven destabilizing forces. Pipelines subject to PGD have suffered severe damages during past earthquakes [1,2]. The damages from PGD can be more severe than those caused by wave propagation damage (WPD). Thus, if relatively a large portion of pipeline is located in a seismically vulnerable area, an accurate estimation of pipe performance due to PGD is more critical than estimating damages caused by WPD. Compared to damages induced from WPD that can spread over a large geographical area, the damages from PGD are concentrated in areas that experience soil liquefaction or landslide. For example, pipeline damages during the 1906 San Francisco earthquake were dominated by PGDs due to widespread liquefaction, such that approximately 52 percent of pipeline breaks occurred within one city block in areas where liquefaction-induced lateral spreading was observed. This area was only 5 percent of the total area impacted by the strong ground shaking [3].

In a regional-scale seismic vulnerability study, pipe damage rates are typically estimated using empirical relationships such as those published in ALA guidelines [4]. Generally, these fragility relationships account for only a limited number of parameters such as the magnitude of PGD, pipe material type and connection type and ignore other influential factors such as the pipe diameter, pipe wall thickness, material strength, burial depth, corrosion, soil parameters, and the direction of ground displacement relative to the pipe axis. Furthermore, most empirical fragility relationships were developed based on the performance of small diameter (less than 300 mm) brittle pipes while the damage data is sparse for large diameter ductile pipes. In this study, an attempt was made to estimate the relative vulnerability of larger diameter welded steel watermains using a pipe-soil interaction (PSI) analysis framework. Although the semi-empirical method relies on some experienced based factors, the impacts from the key influencing parameters are accounted in a fundamental manner. Although the paper focuses only on welded steel watermains, the same formulation can be extended to other utilities.

One of the drawbacks of PSI analysis is that the results are estimates of pipe strain which are not directly linked to a damage rate. The damage rate is often defined as the number of pipe repairs per unit length of pipeline. e.g., the number of repairs per kilometer (or 1000 feet). The damage rate can identify the most vulnerable pipe sections in the network and determine the resource requirements after an earthquake for recovery planning. For example, a system-wide average of only 0.03 “breaks” per 1,000 feet of pipeline (or approximately 0.1 breaks/km) is assigned a serviceability factor of 50 percent in the HAZUS loss estimation tool [5].

## PERMANENT GROUND DISPLACEMENT

### Lateral Spreading

Generally, the largest seismically induced horizontal ground displacements are observed within 250 m from a water body or free face. Beyond this area, the lateral spreading is largely governed by localized topographical features, and displacements generally tend to be small in magnitude and are likely to be shallow. For example, small slumps could occur along drainage ditches and channels and such failures may not be a threat to pipelines buried at a depth deeper than the invert of the drainage ditch. The degree of pipe damage and soil load acting on the pipeline also depend on the shape of the ground deformation profile. The ground deformation profile depends on the variation in soil stratigraphy and is often difficult to estimate with confidence due to uncertainties in liquefaction manifestations. The differential displacements are greatest where buried pipelines transition from liquefiable soils to more competent material. Typically, a rectangular (i.e., abrupt) ground deformation profile is assumed as a conservative worse-case scenario. This assumption is only correct if there is a sudden transition from liquefied to non-liquefied soil or between stable and unstable soil masses in landslide areas. If the ground conditions are uniform, a more gradual change in ground displacement can be considered a more likely assumption. In this paper, the analysis was carried out using the cosine shaped ground deformation profile (i.e., y(x)) proposed by Honegger [6]:

$$y(x) = \delta\left(1 - \left(cos\,\frac{2\pi x}{w}\right)^n\right) \tag{1}$$

Where $x$ is the distance along pipe axis, $\delta$ is the permanent ground displacement, $n$ is an integer that controls the shape of the ground deformation (for this study, $n$ was considered equal to one), and $w$ is the estimated pipeline length over which the ground displacement is occurring.

Historical evidence indicates that the width of a lateral spread PGD zone generally varies from about 75 m (~250 feet) to 600 m (~2,000 feet). For this study, the width of the lateral spreading zone was selected from the statistical analysis completed by Honegger et al. [7] using data from past case histories [8, 9]. The cumulative probability distribution of the width of the lateral spreading zone is shown in Figure 1. Accordingly, the median width of the lateral spreading zone is 100 m. This information is used in the PSI analysis discussed later in this paper.

### Ground Settlement and Uplift

PGD may also occur in the vertical direction due to the post-liquefaction settlement or floatation of the pipeline or buried structure. In flat terrains, the post-seismic free-field settlement is expected to range from about 2 to 5% of the thickness of the liquefiable layer depending on the density of the soil and intensity of seismic shaking. There is some uncertainty related to the liquefaction potential of deeper soil deposits. Even if liquefaction occurs at a deeper depth, it is unlikely to result in a larger differential settlement near the ground surface. Therefore, in most instances, the “free-field” ground settlements triggered from soil liquefaction are unlikely to exceed about 0.5 m. If the pipeline is buried within liquefiable soils, uplift of the pipeline may occur due to excess porewater pressures developed within the liquefiable soil and any loss of strength. Sometimes, it is difficult to distinguish whether a structure was subject to uplift or if the apparent uplift failure was in fact due to the settlement of the surrounding ground. If uplift occurs, the flotation of the pipeline typically results in moderate displacements which are distributed over a large pipe length. Modern pipelines are typically able to accommodate such loading conditions. Even if the pipe itself has adequate resistance to uplift, often the underground chambers are more vulnerable to this buoyancy. If the chamber is subject to uplift, the pipe entering or exiting the chamber will experience relatively large stresses near the transitions.

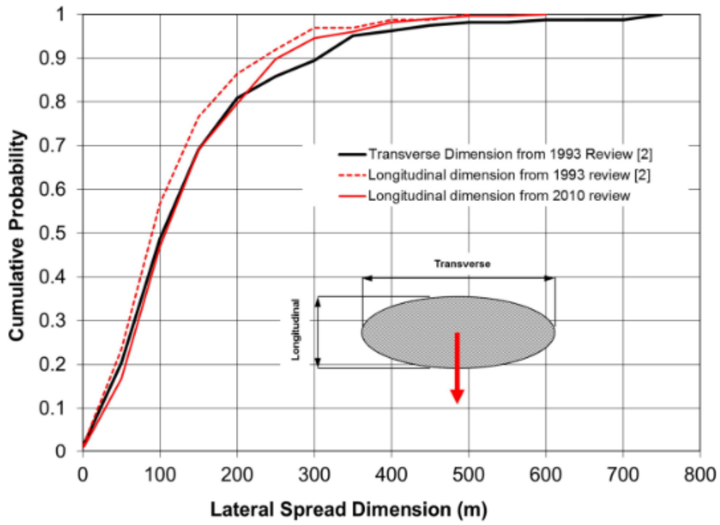



*Figure 1. Width of lateral spreading distribution from past earthquakes (adopted from [7]).*

## FACTORS IMPACTING SEISMIC VULNERABILITY OF PIPELINES

The following discussion describes factors impacting the seismic vulnerability of watermains and how they may be incorporated into a regional-scale assessment.

### Pipe Material and Connections

Pipe material and its connections are critical factors that impact the seismic performance of a pipeline. In past earthquakes, welded steel pipelines using standard construction and quality control procedures adopted in the water industry have shown both relatively high and low damage rates compared to other pipe material types. Although the reasons for vastly different performances are unknown, it is presumed that construction practices, corrosion and geotechnical conditions have contributed. Also, the reported damage rates in these historical events were often controlled by smaller diameter steel pipes (i.e., less than 200 mm diameter pipes used in distribution networks). The data for larger diameter steel watermains is sparse. Examples of the variability in post-earthquake pipeline damage reporting are provided below.

According to [10], during the 1994 Northridge earthquake, nearly 100 repairs (out of 1400) were performed on large diameter transmission and trunk pipes. Out of which, 67 repairs were undertaken in steel trunk pipelines, which experienced excessive deformation or rupture at approximately 34 welded slip joints. A relatively high damage rate was also reported in welded steel pipes during the 1989 Loma Pieta earthquake. However, the post-earthquake analyses have noticed that steel pipes were intentionally deployed in areas with poor ground conditions to improve the network performance [11]. During the 1995 Kobe earthquake, the repair rate of welded steel pipelines was 0.47 failures/km which was considerably lower than the damage rates reported for pipes such as Cast Iron (CI), Polyvinyl Chloride (PVC) and Asbestos Cement (AC) pipes, for which the damage rates ranged from 1.13 to 1.79 failures/km [11]. During the 2010 Chile earthquake, there were 72 failures in large diameter (greater than 500 mm) welded steel pipes [11]. A large-diameter water transmission pipeline located in the epicentral area of the 2011 Tohoku earthquake was damaged at more than 50 locations, mostly due to pulled slip joints. Between September 2010 and December 2011, Christchurch experienced a sequence of strong earthquakes with PGA predominantly ranging from 0.2 to 0.6g [12]. Widespread liquefaction occurred over large areas particularly along Avon River where liquefaction was often associated with relatively large lateral spreading. During this event, 77.5 km out of 1511 km water pipes (5.1% of total pipe length) were damaged. Steel pipes suffered the largest rate of damage at 8.9%, followed by AC pipes and other pipe materials (6.1% and 6.8%, respectively), while PVC (1.8%) and Polyethylene (0.5%) pipes suffered the least damage. However, the sample size for steel pipes was small for a meaningful statistical analysis (only 1.8% of total pipe length); therefore, the results should be treated with caution. Furthermore, most of these steel pipes were likely smaller diameter distribution pipelines. During the 2016 Kumamoto earthquake, steel pipes showed the highest repair ratio among all pipe types with a value of 0.5 failures/km [13], while other pipes such as DI pipes performed extremely well. After segregating the results into different pipe sizes, the highest repair ratio of 3.7 failures/km was observed in steel pipelines with diameters greater than 300 mm.

### Pipe Diameter and Pipe Wall Thickness

For a given pipe material, the axial and bending stiffness of the pipe are influenced by the pipe diameter and pipe wall thickness. Therefore, the pipe diameter and pile wall thickness are recognized as two key parameters that impact the pipe's ability to accommodate ground displacement. Nonetheless, these parameters are not explicitly considered in empirical fragility relationships. Most of the empirical fragility relationships developed prior to 1989 were based on the performance of small diameter (less than 300 mm) CI pipes. Historical earthquakes have not always showed a consistent relationship between damage rates and pipe diameter. For this reason, pipe diameter is not considered as a variable in most of the fragility relationships. During the 1994 Northridge earthquake, there was evidence showing a reduction in damage rates with increasing diameter for CI, AC and DI pipes. The 1989 Loma Prieta earthquake also showed a reduction in damage rates for larger diameter welded steel pipes, although there was no clear indication for CI or AC pipes [13]. Compared to the Northridge earthquake, the correlation was not as strong as for the Loma Prieta earthquake. One explanation given for the observations made during the 1994 Northridge earthquake was that smaller diameter pipes in this earthquake were in the worst soil areas and constructed with low-quality techniques. As a result, the diameter relationship observed in the 1994 Northridge earthquake may not be true for other water systems [4]. According to Shirozu et al. [14], no strong diameter dependency was observed during the 1995 Kobe earthquake where most of the pipe diameters ranged from 100 mm (4") to 300 mm (12"). However, a somewhat lower damage rate was observed when the pipe diameter was 400 mm (16") or larger. There was no distinction made between pipe diameter and soil conditions; thus, it cannot be ruled out that large diameter pipes were in areas with low shaking intensity or smaller PGD. Also, only a few data points were available for larger diameter pipes to conduct a statistical analysis. According to Wham et al. [15], the damage rates of the water supply system following the 2016 Kumamoto earthquake were largely independent of the pipe diameter. However, when the damage to pipe barrel is considered (i.e., ignoring the damages to air valves and other components), the smaller diameter service pipes showed a higher damage rate. Other studies such as those conducted by Sato and Myurata [16] and O'Rourke and Jeon [17] also reported lower damage rates for large diameter pipes.

Ni et al. [18] investigated the pipe diameter effect using numerical modeling. They argued that the flexural strain is a function of the pipe diameter for a specific curvature; therefore, the allowable curvature of a larger diameter pipe is smaller compared to that of a smaller diameter pipe. As a result, a smaller diameter pipe may perform better in PGD areas, as it may be able to tolerate more differential ground movement. Moreover, larger diameter pipes are associated with stiffer/stronger soil restraints (soil springs), such that for the same amount of ground displacement, a larger diameter pipe will experience a larger soil load. For these reasons, Ni et al. [18] stated that an increase in pipe diameter could result in a higher failure probability regardless of whether a constant pipe wall thickness (t) or a constant diameter to thickness (D/t) ratio is maintained. However, there are several reasons why larger diameter pipelines have shown relatively lower damage rates in some earthquakes. For example,

- Damages from earthquakes showed evidence of poor weld quality and presence of corrosion. Smaller diameter pipes receive less attention in terms of weld quality and construction quality control compared to larger diameter pipelines.
- Typically, relatively thin pipe walls are utilized for smaller diameter pipes compared to those used in larger diameter pipelines. As a result, if the corrosion rate is assumed to be constant, the smaller diameter pipes are more vulnerable to damage in an earthquake.
- Most of the failures occur at discontinuities due to stress concentrations. Larger diameter pipes have a fewer number of discontinuities compared to distribution pipelines, making them less vulnerable.
- Often larger diameter pipelines are designed to avoid or mitigate the effects from areas with poor soil conditions. This level of attention is not typically given to smaller diameter distribution pipelines.
- Thicker pipe walls used in large diameter pipes for internal pressure provide greater flexural, tensile and compression capacities.

Considering these factors, ALA [4] recommended to decrease the damage rates by about one-half for lap welded steel pipelines with diameters greater than 300 mm.

### Corrosion and Age

Isenberg [19] reported that over one-half of the leaks attributed to the 1971 San Fernando earthquake were related to corrosion. According to Hakala [20], about 60 percent of pipe breaks and leaks attributed to the 1965 Puget Sound earthquake occurred in steel pipelines, galvanized steel mains and service pipes that were impacted by corrosion. Similar observations were reported in other earthquakes such as the 1969 Santa Rosa earthquake. Steel pipes have relatively thinner walls than CI or DI pipes; therefore, they are more vulnerable to failure from localized pitting corrosion. Age and corrosion also accentuate damages in other pipe material such as CI pipes (e.g., [19]). For example, in the 1983 Coalinga earthquake, O'Rourke and Ayala [21] attributed the unusually high damage rate in CI pipes to corrosion. Corrosion is not expected to be a significant factor if corrosion protection measures have been implemented. While the pipes without corrosion protection are more susceptible to failure, the actual likelihood will depend on exposure conditions (e.g., pH, soil resistivity and soil aeration). It is typically not possible to incorporate such details in a regional-scale study, unless records are maintained by the utility owner on repairs and extent of corrosion.

When collecting data to develop fragility-based relationships, the status of the pipeline with respect to corrosion is often overlooked. Therefore, the damage rate is an overall rate that includes pipes with different levels of corrosion impact. ALA [4] stated that damage rates for a small diameter steel pipe in a corrosive soil is about 50 percent greater; while it is expected to be about 50 percent less if the pipe is not impacted by corrosion. This infers that the damage rate between a pipe impacted by corrosion is three times greater than a pipe not impacted by corrosion. There are some uncertainties related to the applicability of this factor for larger diameter pipes due to lack of damage data for such pipes. Corrosion of a larger diameter pipeline may manifest as a smaller pin hole leak; therefore, it may not be as pervasive as it is for a smaller diameter pipe. Often the age of the pipeline is strongly correlated to its state of corrosion since the impact of corrosion increases with time. Typically, relatively new steel pipes (i.e., 25 years old or less) will not be significantly impacted by corrosion even if they are in a corrosive environment compared to older steel pipes in the same environment. Eidinger [22] observed that older pipelines have a higher damage rate than newer steel pipelines in earthquakes such as the 1987 Whittier Narrows and 1989 Loma Prieta earthquakes. In the PSI based method discussed in the subsequent sections, the corrosion impact is accounted for using the $K_c$ factor. The $K_c$ factors selected for lap welded large diameter steel pipes are shown in Table .

*Table 1: Modification factors selected for large diameter lap welded steel pipes*

| **Corrosion Impact ($K_c$)** | **Weld Type ($K_A$)** | |
|---|---|---|
| | **Before 1935** | **After 1935** |
| $K_c$ = 1.0 (if cathodically protected, regardless of the age) | | |
| $K_c$ = 1.0 (constructed before 1935). | | |
| $K_c$ = 3.0 (constructed after 1935 and more than 50 years old) | $K_A$ = 5 | $K_A$ = 1 |
| $K_c$ = 1.5 (between 25 to 50 years old) | | |
| $K_c$ = 1 (less than 25 years old) | | |

### Type of Weld Connection

Welded steel pipelines installed prior to mid-1930s were often joined by a single-pass oxyacetylene gas weld. This type of weld has a large heat-affected zone and potentially higher carbon content compared to an arc welded or a multi-pass oxyacetylene weld [4]. For example, the 1933 Long Beach earthquake caused more than 50 breaks in high-pressure gas pipelines. Each break was identified at welds that lacked the proper penetration or bond with the pipe body. This aspect is not explicitly considered in ALA fragility relationships. Nonetheless, ALA [4] recommends that welded steel pipe with arc-welded joints to be considered as ductile while gas-welded joints be treated as brittle. For a seismic vulnerability assessment conducted for Seattle, WA, USA, Ballantyne et al. [23] estimated a damage rate of 0.5 failures/km for welded steel pipelines with arc-welded joints in areas impacted by PGD. Comparatively, the damage rate was 2.4 failures/km for welded steel pipes with oxyacetylene welds based on historical damage rates. Based on this, it was decided to increase the damage rate by a factor of five for pipes constructed prior to 1935 to account for potential oxyacetylene gas welds in the pipe. This impact of weld connection type on the damage rate (or age of the pipe) is represented by $K_A$ factor as shown in Table 1. Corrosion was not explicitly considered for pipes constructed prior to 1935, since the factor to account for the weld type would have already included any potential impact from aging/corrosion.

### Discontinuities in Pipe Network

Pipeline damages often tend to concentrate at discontinuities such as pipe elbows, tees, in-line valves, reaction blocks and service connections. Such components would create anchor points or rigid locations that promote force/stress concentrations. For example, the liquefaction-induced ground movements during the 1971 San Fernando earthquake caused severe damage to a 1245 mm (49 inch) diameter water pipeline at nine bends and welded joints [1]. Locally high stresses can also occur at pipeline connections to adjacent structures (e.g., tanks, buildings and bridges), especially if there is insufficient flexibility to accommodate relative displacements between the pipe and structure. During the 2016 Kumamoto earthquake, Ishida [13] reported the extent of damage to air valves. The damage rate at air valves was significantly higher compared to the damages to the pipe itself (e.g., the damage rate for air valves was 0.05 failures/km compared to the total damage rate of 0.14 failures/km).

Similar damages to air valves have been observed during the 2011 Tohoku earthquake. These damages have occurred not only in areas near the earthquake epicenter but also in areas far from the epicenter, suggesting that the cause of damage is not related to the strong seismic shaking. Most of the damages were associated with the float valve body. Therefore, it was postulated that abrupt increase of water pressure from sudden closure of valves (triggered from the earthquake) led to these failures. This form of component vulnerabilities is difficult to model using PSI analysis. However, the empirical fragility relationships are likely to incorporate such damages if these failures were identified in the database used for the development of these relationships.

### Soil Load Acting on Pipe

The soil load acting on the pipe from PGD will depend on several factors as described below. Soil restraints for numerical analysis can be represented using PRCI guidelines [5]. Note that none of these factors are explicitly considered in available fragility-based relationships except for the magnitude of PGD.

- **Magnitude and shape of ground displacement:** Inevitably, the magnitude of ground movement is one of the key factors that influences the pipeline failure. It is not always possible to quantify the damage due to ground movement even with sophisticated analysis because of the uncertainties related to the ground movement estimations and difficulties in accounting for the complex interactions that may occur at pipe bends and other discontinuities.
- **Pipe burial depth:** In areas susceptible to earthquake-induced ground movement, pipe burial depth ($H$) has a significant influence on the soil load acting on the pipe since the stiffness of the soil spring is a function of the $H/D$ ratio. For example, for the same magnitude of ground displacement, a pipe with a shallower burial depth will experience a smaller soil load. The exact burial depths are difficult to incorporate into a regional scale assessment.
- **Backfill soil conditions:** The soil load acting on the pipe is impacted by the density and strength of surrounding soils. A denser soil will exert a larger soil load than a looser soil backfill. Details of backfill soil are not readily available and often ignored in regional-scale seismic studies. However, large diameter transmission pipelines are typically located along existing roads and surrounded by compacted granular fill during installation; therefore, it is generally considered reasonable to conduct PSI analysis assuming dense granular backfill. This study assumed a friction angle of 40 degrees and a soil density of 19 $kN/m^3$.

### Built-In Pipe Stresses

Some pipelines may be impacted by natural and man-made causes. For example, pipelines can be impacted by long-term settlement if they are underlain by compressible soils such as peat/organic, silts and clays. Additional loading from earthquakes will exacerbate the damage to these pipelines. Such effects are not explicitly considered in empirical fragility-based relationships. However, in the PSI analysis discussed in the following sections, the $K_S$ factor is introduced to account for the built-in pipe stresses, if the pipeline is known to be located in an area that is prone to long-term settlement or other factors.

## PIPE CAPACITY

### Joint Capacity

Typically, steel pipes consist of lap welded (bell and spigot) joints. This type of joint is commonly used since it is a simple and efficient way of connecting large-diameter thin-wall pipe segments. The field welded double or single lap joints are considerably weaker compared to the pipe barrel; thus, the overall pipeline performance is significantly impacted by the performance of joints under compression (local buckling or wrinkling) or tensile loads. The strain capacity of a lap welded joint is influenced by pipe wall thickness, bell geometry and eccentricity between spigot outside and bell inside diameters. The eccentricity of the lap welded joint introduces additional stresses at the joint and reduces its capacity to accommodate the ground movement [3]. For both tensile and compression failure modes, the most common failure mode is the rupture resulting from circumferential cracking of the pipe material at the bell joint or at the bell-to-spigot weld.

Large compression forces could lead to the fracture or crushing of welded slip joints. The compressive strain limit of steel pipes is primarily governed by the D/t ratio. The joint efficiency of a single lap welded joint is approximately 40 to 45 percent. The joint efficiency of a double lap welded (bell and spigot) joint is not signifcantly different to a single lap welded joint, and is estimated to be about 55 percent [24]. In comparison, a full penetration butt weld will have a joint efficiency excceeding 95 percent. For a double lap welded joint, Dorey et al. [25] recommended the compression capacity be limited to the following, regardless of the pipe class:

$$\varepsilon_{cp} = 1170\left(\frac{t}{D}\right)^2 \tag{2}$$

For a single lap welded joint, the compression capacity is limited to 40 percent of the yield stress. Most of the research on weld joint capacity have been focused on pure tensile or compression loading modes while the mechanical behavior of those joints under bending has not been thoroughly investigated.

Typically, the tensile failure of a welded-slip joint occurs at strains greater than 2 percent, which indicates that those joints can sustain a considerable amount of inelastic deformation before failure. The allowable strain of a lap welded joint is smaller than the ultimate strain. For example, for a single lap welded joint, Metro Vancouver Seismic Design Criteria [26] recommended to limit the tensile strain of a Class IV pipe to 40 percent of the yield strength ($\sigma_y$) while it can be increased to the yield strength for a double lap welded joint.

**Pipe Capacity**

When the compressive strain exceeds a certain threshold, local buckling or wrinkling of the pipe wall will occur. A water pipeline may still be able to fulfill its basic functions even after local buckling or wrinkling [28]. Significantly higher strains are required before wrinkling or buckling impede the passage of in-line inspection or cleaning devices. According to [26], the compression strain limit ($\varepsilon_{cp}$) is taken as:

$$\varepsilon_{cp} = 1.76\ \frac{t}{D} \tag{3}$$

For a displacement-controlled failure mode such as PGD, the pipe's tensile strain capacity can exceed the yield strain by a considerable amount. In such situations, the allowable tensile strain for a strain-controlled loading scenario can be conservatively assumed to be equal to about 6 to 8 percent under bi-axial loading conditions. This shows that tensile rupture of the pipe barrel is unlikely to occur because the tensile rupture of the weld joint is likely to occur initially.

**Pipe Failure Types**

Typically, a "leak" is defined "*as a pipeline failure where a pipeline is losing its product but might continue to operate until the leak is detected*, whereas a "break" is defined as a "*pipeline failure where a pipeline cannot continue to operate".* As expected, a break may lead to more severe consequence than a leak. Despite the differences, leaks and breaks have been collectively called "failures" or "damage" in this paper. Generally, when a pipeline is damaged from PGD, the damage is more severe (i.e., more breaks). In contrast, the damages from WPD which are typically caused by joint pull-out or crushing at the bell and are likely to cause more leaks than breaks. The fragility relationships have been derived from pipe repair databases developed from historical earthquakes. Often the repair records include details such as type of repair, location and time, but do not always include sufficient information to determine the severity of the damage. For damages predicted using WPD, [5] recommends 80 percent of damages be treated as leaks while the remaining 20 percent is considered as breaks. The repair data from the 1949 and 1969 Seattle, 1969 Santa Rosa, 1971 San Fernando Valley, 1983 Coalinga, and 1987 Whittier Narrows earthquakes shows approximately 15 percent of all repairs have been identified as breaks [4], which is somewhat consistent with the recommend split between leaks and breaks in [5]. For PGD induced damages, FEMA [5] suggests considering 80 percent of damages as breaks, while the remaining 20 percent is treated as leaks. There is significant uncertainty related to this breakdown. In the aforementioned earthquakes, approximately 50 percent of repairs were identified as breaks when the damage was caused by PGD [4]. The 80/20 split recommended in [5] is likely to have been influenced by brittle segmented pipelines; therefore, the division is likely to overestimate the breaks for continuous welded steel pipelines that require a relatively large strain to cause a break/rupture in the pipeline. In this paper, only the total damage rate was estimated and the split between leaks and breaks becomes important in the next phase when determining the required resources and recovery periods.

## ESTIMATION OF DAMAGE RATES FROM PSI

This paper only discusses the method of evaluating the number of leaks and breaks in the pipe system due to horizontal and vertical ground displacements occurring perpendicular to the pipe axis. As stated earlier, PSI analysis will provide an estimation of the pipe demands (bending moments, shear forces, and axial forces) but not the damage or repair rates. Therefore, the following relationship was developed to convert the estimated pipe demands to a repair rate ($RR_{PGD}$), in repairs/kilometer.

$$RR_{PGD} = K_A \times K_s \times K_c \times \frac{1000}{L_1} \times \left[\sum_i P_{c,i}\left(\frac{d_i}{L_1}\right)\left(\frac{d_i}{L_2}\right) + \frac{d_i}{L_1} \times P_{p,i}\right] \tag{4}$$

Where:

$L_1$ = Pipeline length in question (m). This was selected as 300 m if the pipeline was located 250 m from a waterbody. As indicated previously, the median width of the lateral spreading zone is 100 m. If so, approximately 33 percent of the pipeline is assumed to be impacted by lateral spreading. This is considered reasonable and consistent with previous regional seismic studies conducted for FortisBC [7]. For example, in the 1994 FortisBC study [7], areas within 1 km (compared to 300 m assumed in this study) of major river channels were assumed to have a 34 percent chance of experiencing lateral spread displacement. Away from the waterbody, this length can be increased if more uniform ground displacements are expected.

$L_2$ = Pipeline length between two joints (m). The pipeline segment length typically ranges from 6 m (20 feet) to 12 m (40 feet). The intent of including this term is to estimate the probability of encountering a weld within the zone of high bending strain (=$d_i/L_2$).This PSI analysis was based on a pipeline segment length of 6 m.

$K_c$ = Factor to account for corrosion.

$K_s$ = Factor to account for built-in stresses in pipelines (e.g., due to long-term settlement of soil).

$K_A$ = Factor to account for the welding type (and age). As discussed earlier, the damage rate was increased by applying a $K_A$ factor of five to account for the potential weak welds constructed prior to 1935 using a single-pass oxyacetylene gas weld. This was a simplification in the analysis method instead of adjusting the failure probability of the weld connection.

$P_{p,i}$= Probability of failure of the pipe itself for a given range of pipe strain. The failure probabilities selected for the pipe are given in Table 2. The strain limit was selected to represent the compression loading mode which is more severe than tension for thin wall steel pipelines experiencing bending. The failure probabilities are largely based upon subjective judgments. It is noted that the values are somewhat consistent with the judgment-based values selected for the FortisBC seismic vulnerability study that were conservatively biased owing to the level of consequences in the event of gas pipeline rupture [7].

$P_{c,i}$ = Probability of failure of a lap welded connection for a given range of strain. The failure probabilities of weld slip joints were selected after considering the relative vulnerabilities of the pipeline and weld connection (see previous discussions regarding joint efficiencies). These failure probabilities are applicable to both single and double lap welded joints since their performance is not significantly different. For pipelines experiencing bending, the capacity of the weld will be governed by compression; therefore, the failure probabilities are given only for this mode of failure.

$d_i$ = Pipeline length that falls within a certain range of pipe strain (m).

*Table 2: Failure probabilities of pipe body and single or double slip weld joints for different bending strains (compression)*

| **Pipe** | | **Weld Connection** | |
|---|---|---|---|
| **Bending Strain (Compression)** | **Probability of Failure** | **Bending Strain (Compression)** | **Probability of Failure** |
| Less than $\sigma_y$ | 0% | Less than min (0.4$\sigma_y$, 1170 (t/D)$^2$) | 0% |
| $\sigma_y$ to 1.76 t/D | 10% | Min (0.4$\sigma_y$, 1170 (t/D)$^2$) to $\sigma_y$ | 20% |
| 1.76 t/D to 2 times 1.76 t/D | 50% | $\sigma_y$ to 2 times $\varepsilon_y$ | 50% |
| Greater than 2 times 1.76 t/D | 100% | Greater than 2 times $\varepsilon_y$ | 100% |

The non-linear stress-strain behavior of the pipe material was considered as it provides a more realistic response of the pipeline when the strain exceeds the yield strain of the pipe. As an example, Figure 2 shows the bending moment – curvature and bending moment- bending stiffness relationships derived for a 914 mm x 6.4 mm steel pipe with a yield strength of 248 MPa.

The PSI analyses were conducted using the LPILE [29] program. Although this program was developed for analyzing piles, the analytical formulation relies on the same Winker beam theory used for PSI. Furthermore, LPILE allows user-defined soil springs and ground displacements to be incorporated. Compared to some of the traditional programs used for PSI analysis, the greatest benefit of LPILE is the user-friendly interface that allows multiple scenarios to be analyzed with relative ease - a crucial aspect in a regional scale seismic study. LPILE can consider only ground movements occurring perpendicular to the pipe axis and cannot account for complex ground movements (e.g., oblique to the pipe), nor the beneficial effects of pipeline tension under large lateral displacements. Furthermore, the program does not have the ability to apply any internal pressures from the fluid inside the pipeline. The error arising from these limitations is not significant for a regional scale analysis compared to the uncertainties associated with the empirical relationships.

Figure 3 shows the pipe deformations and pipe strains estimated for a 914 mm x 6.4 mm pipe buried at a depth of 1.8 m. The horizontal ground displacement is 0.75 m and the width of PGD was 100 m. A steel pipe with a yield strength of 248 MPa was considered. The red, orange, and green areas shown in Figure 3 represent the different strain thresholds considered in Table 2 for lap welded slip joint. Both cosine and abrupt ground deformation shapes were considered in the PSI analysis. In this case, the cosine-shaped ground displacements were found to provide the highest damage rate although the abrupt ground displacement resulted in the largest pipe strain. For an abrupt ground movement, 19.2 m of pipe length ($d_1$) is within 0.05 to 0.12 strain range, 10.8 m of pipeline length ($d_2$) is within 0.12 to 0.24 strain range and 8.4 m of pipeline length ($d_3$) has strains greater than 0.24. Assuming, $K_S = 1$, $K_C = 1$, $K_A = 1$ and $L_2 = 6$ m, the estimated $RR_{PGD}$ is 0.41 damages/km. As the pipe yields, the highest strain area is limited to a narrow pipeline length, possibly isolating other pipe sections from damage. In contrast, the cosine shaped ground displacement profile results in a smaller strain but it is distributed over a longer pipeline length. In this example, a 43.2m long pipeline section will experience strains between 0.12 and 0.24. This results in a slightly higher repair rate of 0.69 repairs/km, which is calculated as follows: $RR_{PGD} = 1 \times 1 \times 1 \times \frac{1000}{300} \times \left[0.2\left(\frac{43.2}{300}\right)\left(\frac{43{,}2}{6}\right) + \frac{43.2}{300} \times 0\right] = 0.69$. The first term within brackets represents the probability of failure at the weld connections and the second term represents the probability of failure of the pipe itself. In this case, as the estimated pipe strain does not exceed 0.12 for the cosine shaped ground displacement, no failures are predicted in the pipe barrel. If the frequency of weld connections is increased from every 6m to every 10m, the failure rate will decrease to 0.41 repairs/km from 0.69 repairs/km.

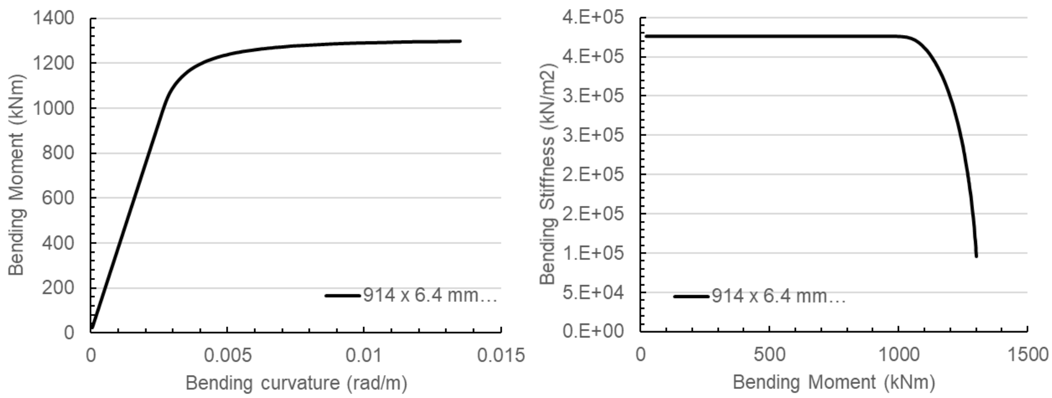



*Figure 2: Bending moment, curvature and bending stiffness considered for a 914 mm x 6.4 mm steel pipe.*

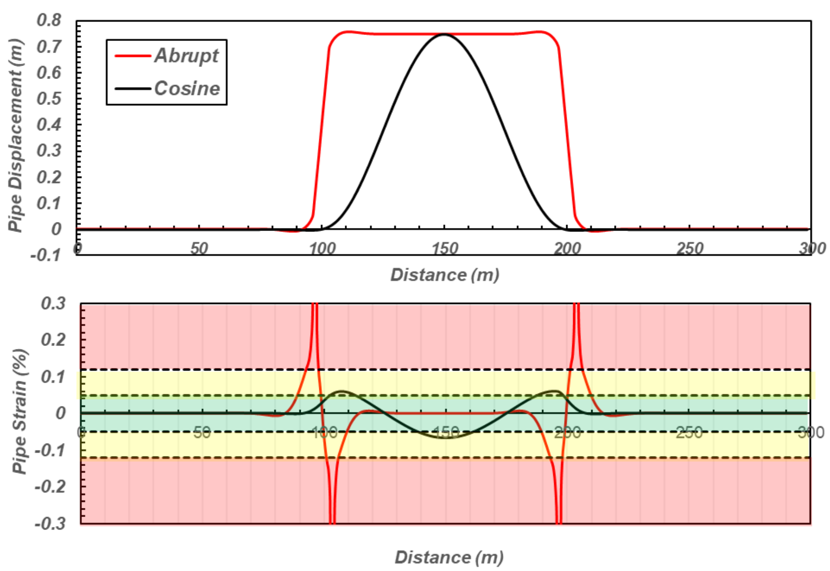



*Figure 3: Pipeline deformation shapes and bending strains for 0.75 m of lateral spreading (914 mm x 6.4 mm steel pipe). As a reference, the coloured zones represent the strain ranges given in Table 2 for weld connections.*

The analysis was repeated for several pipe diameters and different horizontal ground displacements. The buried depth was selected such that an *H/D* ratio of two was maintained for all pipe diameters. The pipe diameters, pipe wall thicknesses and pipe burial depths considered in this assessment are summarized in Table 3. The estimated damage rates for different pipe diameters are shown in Figure 4(a) and 4(b). The soils springs were estimated based on the guidelines presented by Pipeline Research Council International [6]. Damage rate is the largest damage rate predicted using cosine or abrupt ground displacement profiles. Some of the key observations are:

- For smaller diameter pipelines, the failure rates are insensitive to the ground displacement magnitude. This is due to the yielding of the pipe that isolates the remaining pipeline sections from damage.
- At smaller ground displacements and in smaller diameter pipes, abrupt ground displacements are more critical than the cosine shaped ground displacements. This behavior reverses as the pipe diameter and ground displacement are increased. This is due to the relatively high flexural rigidity of larger diameter pipes that causes bending moment to be distributed over a longer section of the pipeline. This in turn will expose more weld connections to damage.
- The failure rate increases as the ground displacement increases although the increase is larger when the pipe diameter increases. It should be reminded that the same wall thickness (i.e., 6.4 mm) was assumed for every pipeline; hence, the larger diameter pipes are associated with a larger *D/t* ratio. A smaller damaged rate is expected if wall thickness is increased.

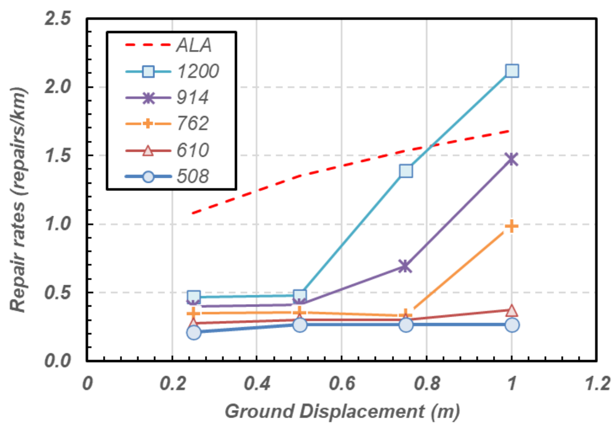
For comparative purposes, the damage rates estimated using the following empirical relationship proposed in ALA guidelines [4] are also shown in Figure 4(a):

$$RR_{(PGD)} = \mathrm{K}_2 \times 1.06 \times \mathrm{PGD}^{0.319} \tag{5}$$

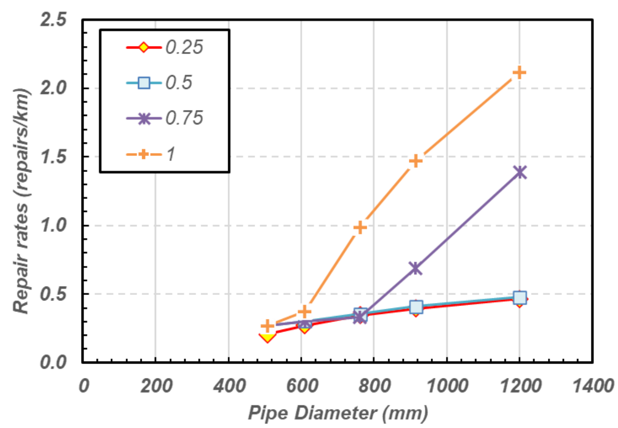
Where $DR_{(RGD)}$ is the repair rate for pipeline from PGD (repairs/1000 feet) and PGD is in inches. $K_2$ is a factor that account for the pipe material type, connection and pipe diameter. Although ALA relationship appears to predict generally a higher damage rate, it is reminded that the PSI based method do not account for any adverse effects from corrosion, aging (or weld type) and built-in stresses in the pipeline (i.e., $K_S$, $K_C$ and $K_A$ factors are equal to one).

*Table 3: Pipe diameters, pipe wall thicknesses, and burial depths considered for pipe-soil interaction analysis*

| **Pipe Diameter, D (mm)** | **508** | **610** | **762** | **914** | **1200** |
|---|---|---|---|---|---|
| Pipe Wall Thickness, t (mm) | 6.4 | 6.4 | 6.4 | 6.4 | 6.4 |
| Pipe Burial Depth, H (m) | 1.02 | 1.22 | 1.52 | 1.83 | 2.4 |




(a)




(b)

*Figure 4: Estimated repair rates using the PSI based approach as a function of (a) different ground displacements and (b) pipe diameters*

This PSI-based formulation allows to differentiate the pipeline performances in areas experiencing lateral spreading and post-seismic settlements. Due to the differences in soil springs in respective directions, the failure rates are not necessarily similar even if the magnitude and shape of the ground displacement are the same. In contrast, empirical methods such as ALA [4] do not consider the direction of loading; therefore, the same damage rate is estimated in the horizontal and vertical directions.

At the start of the regional level study, a series of PSI analysis should be performed to develop repair rates for different ground movement magnitudes, directions (horizontal and vertical) and different shapes. Besides using the actual pipe diameter and connection types, the accuracy can be improved if the analysis is performed using the most representative burial depths and pipe wall thicknesses encountered in the areas where PGD are anticipated. Once these relationships are developed, the damage rates can be adjusted to account for the age of the pipeline (or type of weld), corrosion impact and built-in stresses from soil settlement and other factors. In addition, any changes to the areal distribution of the pipeline within the liquefiable area can be included by using the $L_1$ parameter (pipeline length in question). This is a key parameter that can impact the estimated failure rate. For example, if $L_1$ is increased from 300 m to 400 m, the repair rate will decrease from 0.69 repairs/km to 0.39 km/repairs for the 914 mm diameter pipe discussed previously. In essence, $L_1$ is an indicator of how close the next PGD feature would manifest. Some judgement is required in selecting a suitable value for $L_1$, possibly based on observations from historical earthquakes. A relatively shorter $L_1$ distance may be considered closer to waterbodies where ground displacement features are prominent. Beyond this area, it may be reasonable to increase $L_1$. This is a consideration in areas such Richmond and Delta in the Lower Mainland, where large areas are expected to undergo soil liquefaction although most of the severe ground displacements are likely to occur near Fraser River.

## CONCLUSIONS

Owing to their simplicity, empirical methods are advantageous when analyzing distribution systems with a large number of small diameter pipelines, and often, there is not a significant benefit in implementing the PSI based approach for such pipe networks. The proposed PSI based approach is beneficial for transmission pipeline networks where historical data is sparse from which to develop reliable empirical relationships. Furthermore, these pipelines include fewer discontinuities in the system and total pipeline length, number of pipe materials, and other variables that are considerably less than those for a distribution network. In a regional scale analysis, it is not feasible to conduct site-specific PSI analysis for complex pipeline configurations or at every discontinuity to assess the vulnerability of these locations. The additional effort required to conduct a PSI based approach is justifiable for transmission pipeline networks as the estimated damage rates account for key factors that can impact the pipeline performance in a fundamental manner. Despite the benefits of the PSI based method, it is not possible to develop the method entirely on an analytical framework since some factors are based on past experience/data. For these reasons, if the damage rates predicted from the PSI based method are significantly different to those estimated using empirical methods, further scrutiny should be given to identify the reason for the discrepancy and to assist in selecting the most appropriate damage rate.

As stated earlier, PSI occurring along the pipe axis is not discussed in this paper since a different formulation is required for PGD in this direction. Even for horizontal and vertical ground displacements, only select cases are discussed in this paper owing to the page limitations. Within the constraints of the numerous assumptions employed in defining the PSI fragility functions, some of the key observations are summarized below.

Similar to ALA fragility curves, damage rates are expected to increase with the amount of ground displacement. However, the increase will depend on the pipe diameter and shape of the ground deformation profile. Damage rate is insensitive to ground displacement magnitude if yielding occurs in the pipe. This is more likely to occur in small diameter pipes and for cases when the ground displacement is abrupt.

PSI analysis indicates that the damage rate will increase with the pipe diameter. However, in this study, D/t ratios are not the same for each pipe size; therefore, different results should be expected for different D/t ratios. As stated earlier, better construction and quality control measures adopted for larger diameter pipes are not reflected in the PSI analysis, although they can have an impact on the relative vulnerability.

A key item of information extracted from the PSI analysis is the relative vulnerability of the pipe barrel and weld connection. According to the PSI based approach, the number of damages expected in the pipe and weld can be calculated separately, which is not possible using fragility relationships. Although it is difficult to generalize, for larger diameter pipes (i.e., larger than 762 mm), the PSI analysis indicates that almost all failures would occur in the pipe barrel. This observation is applicable for cosine shaped ground displacements which resulted in the largest damage rate compared to an abrupt movement. However, in smaller diameter pipelines (e.g., less than 762 mm), about 60 to 70 percent of failures would occur at weld connections while the remaining failures would occur in the pipe barrel. The failure rate at connections was on the order of 60 percent during the 1994 Northridge earthquake and 80 percent during the 1995 Kobe earthquake. During the 2016 Kumamoto earthquake, in pipelines having diameters larger than 600 mm, only about 10 percent of the damages occurred in the pipe barrel. This indicates that PSI predictions are somewhat consistent with observations from actual earthquakes, suggesting that relative failure probabilities selected for pipe and weld connections are reasonable.

## REFERENCES

[1] O'Rourke, T.D. and Tawfik, M.S. (1983). "Effects of Lateral Spreading on Burled Pipelines During the 1971 San Fernando Earthquake", *In Symposium on Lifeline Earthquake Engineering, Earthquake Behavior and Safety of Oil and Gas Storage Facilities*, Buried Pipelines and Equipment, PVP-Vol. 77, American Society of Mechanical Engineers (ASME).

[2] O'Rourke, M.J. (1989). "Approximate Analysis Procedures for Permanent Ground Deformation Effects on Burled Pipelines", In *2nd U.S. Japan Workshop on Liquefaction, Large Ground Deformation and Their Effects on Lifelines*, Buffalo, N.Y., September 1989 (NCEER Report No. NCEER-89-0032).
[3] O'Rourke, M.J. and Liu, X. (1999). *Response of buried pipelines subject to earthquake effects*. Multidisciplinary Center for Earthquake Engineering Research, University of Baffalo, Buffalo, NY., USA.
[4] ALA. (2001). *Seismic fragility formulations for water systems: Part 1: Guidelines*. American Lifelines Alliance, American Society of Civil Engineers, Reston, VA., USA.
[5] FEMA. (2020). *Hazus technical manual*, Federal Emergency Management Agency, Washington, D.C., USA.
[6] Honegger, D. G. (2017). *Pipeline Seismic Design and Assessment Guideline*. Pipelines Research Council International.
[7] Honegger, D.G., Wijewickreme, D., and Youd, T.L. (2014). “Regional Pipeline Vulnerability Assessment Based Upon Probabilistic Lateral Spread Hazard Characterization.” In *10th National Conference on Earthquake Engineering*, July 21-25, 2014, Anchorage, Alaska, USA.
[8] Hamada, M. and O'Rourke, T. D. (1992). *Case studies of liquefaction and lifeline performance during past earthquakes.* National Center for Earthquake Engineering Research, Buffalo, NY, USA.
[9] Hamada, M., Isoyama, R. and Wakamatsu, K. (1995). *The 1995 Hyogoken-Nambu (Kobe) earthquake liquefaction, Ground displacement and soil condition in Hanshin area*. Association for Development of Earthquake Prediction.
[10] O'Rourke, T. D., Toprak, S. and Sano, Y. (1998). “Factors affecting water supply damage caused by the Northridge earthquake”. In *the Sixth U.S. National Conference on Earthquake Engineering*, May 31-June 4, 1998, Seattle, Washington, USA.
[11] Ballantyne, D. (1997). “*Performance of water systems in historic earthquakes*”, Reliability and Restoration of Water supply Systems for Fire Suppression and Drinking Following Earthquakes, edited by D.B. Ballantyne C.B. Crouse, prepared for the National Institute of Standards and Technology, Gaithersburg, Maryland.
[12] Bradley, B. A. and Cubrinovski, M. (2011). “Near-source strong ground motions observed in the 22 February 2011 Christchurch earthquake”. *Seismological Research Letter*, 82 (6), 853–865.
[13] Ishida, A. (2015). *Study on earthquake damage factor of air valve in water distribution network of water supply*, Master’s Thesis, Kanazawa University.
[14] Eidinger, J. (1998). *Lifelines, water distribution system in the Loma Prieta, California, earthquake of October 17, 1989*, Performance of the Built Environment -Lifelines, US Geological Survey Professional Paper 1552-A, pp A63-A80.
[15] Shirozu, T., Yune, S., Isoyama, R., and Iwamoto, T. (2000). “Report on damage to water distribution pipes caused by the 1995 Hyogoken-Nanbu (Kobe) earthquake.” In *6th Japan-U.S. Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures Against Soil Liquefaction*, NCEER Report NCEER-96-0012.
[16] Wham, B.P, Pariya-Ekkasut, C., Argyrou, C., and Lederman, A. (2017). “Experimental characterization of hazard-resilient ductile iron pipe soil/structure interaction under axial displacement”. *In Congress on Technical Advancements*, Duluth, MN, USA, 10–13 September, 2017.
[17] Sato, R and Myurata, M. (1990). *GIS-based interactive and graphic computer system to evaluate seismic risks on water delivery networks.* Department of Civil Engineering and Operations Research, Princeton University. New Jersey, USA.
[18] O’Rourke, T. D. and Jeon, S. S. (1999). “Factors affecting the earthquake damage of water distribution systems". In *the Fifth U.S. Conference on Lifeline Earthquake Engineering*, Seattle, Washington, ASCE, Reston, VA, 379-388.
[19] Ni, P., Mangalathu, S., and Yi., Y. (2018). “Fragility analysis of continuous pipes subjected to transverse ground displacement”. *Soils and Foundations,* 58(6), 1400-1413.
[20] Isenberg, J. (1979). "Role of corrosion in water pipeline performance in three U.S. earthquakes". In *the Second U.S. National Conference on Earthquake Engineering,* Earthquake Engineering Research Institute, August 1979, pp. 683-692.
[21] Hakala, W. (1981). *Seismic design of water and wastewater systems*. Prepared for Environment Quality Systems, Inc. Rockville, Maryland., USA.
[22] O'Rourke, M.J. and Ayala, G. (1990). "Seismic Damage to Pipeline: Case Study", *Journal of Transportation Engineering*, American Society of Civil Engineers, 116(2), 123-134.
[23] Eidinger, J. (1998). *Water Distribution System – The Loma Prieta, Californa, Earthquake of October17, 1989 - Lifelines*, USGS, Professional Paper 1552-A, Anshel J. Schiff (ed.), U.S.Government Printing Office, Washington, A63-A78.
[24] Ballantyne, D., Berg, E., Kennedy, J., Reneau, R., and Wu, D. (1990). *Earthquake loss estimation modeling of the Seattle water system*. Prepared by Kennedy/Jenks/Chilton.
[25] Brockenbrough, R.L. (1990). “Strength of bell and spigot joints”. *Journal of Structural Engineering*, ASCE, I16(7), https://doi.org/10.1061/(ASCE)0733-9445(1990)116:7(1983)
[26] Dorey, A.B., Cheng, J.J.R. and Murray, D.W. (2001). *Critical Buckling Strains for Energy Pipelines*. Structural Engineering Report 237, Department of Civil Engineering, University of Alberta, Canada.
[27] Metro Vancouver (2015). *Seismic design criteria*. Revision D, Metro Vancouver Engineering Standards, April, 2015.
[28] Gresnigt, A. M. (1986), “Plastic design of buried steel pipelines in settlement areas”, In *International Offshore and Polar Engineer Conference (ISOPE)*, Stavanger, Norway.
[29] Ensoft. (2019). *LPILE technical manual*: *A program for the analysis of deep foundations under lateral loading*.