

# ROLE OF GEOTECHNICAL ENGINEERING IN ASSURING THE INTEGRITY OF BURIED PIPELINE SYSTEMS

Dharma Wijewickreme<sup>1</sup>

<sup>1</sup>University of British Columbia, Vancouver, B.C., Canada  
Telephone: 1-604-822-5112; Fax: 1-604-822-6901  
E-mail: [dharmaw@civil.ubc.ca](mailto:dharmaw@civil.ubc.ca)

## Abstract

Buried pipeline systems form a major part of global infrastructure that threads the human-made physical environment, contributing to the health, safety, and welfare of communities. Satisfactory performance of these pipeline systems, therefore, has become a key factor in assuring a sustainably built environment since any significant disruption to them often translates into undesirable impacts on economies or the living conditions of citizens. Geotechnical engineering has a dominant role to play in ensuring satisfactory performance of buried pipelines. Adequate knowledge of site-specific soil and groundwater conditions is critical to the design of pipelines, as well as ensuring good predictions of their field performance. Quantification of anticipated geotechnical hazards and evaluation of their impacts are other important considerations in assessing the long-term performance of buried pipelines. The interaction between buried pipes and surrounding soil is complex. Therefore, approaches ranging from those based on simplified assumptions to sophisticated numerical modeling techniques need to be employed in solving soil-pipe interaction problems. The overall goal is to reduce the risk of damage to buried pipelines from geotechnical hazards. A spectrum of options such as isolation from the hazard, accommodation of the hazard, or mitigation of the hazard using ground improvement can be considered in this regard.

**Keywords:** Buried pipelines, pipeline integrity, geotechnical engineering, geotechnical hazards

# 1. Introduction

Buried pipeline systems form a key part of global lifeline infrastructure that serves as the backbone to the human-made physical environment that contributes to upholding the health, safety, and welfare of society. Pipeline systems are commonly used to transport large quantities of fluids between geographic locations. Compared to ground transportation, they offer a mode of transfer of fluids at lower cost per unit volume and at higher capacity; the transportation of water, sanitary waste, oil, gas, and hot water (for district heating) are some examples in this regard. Most pipelines are located along over-land alignments although there are some major pipelines that are located under the sea primarily to transport petroleum products.

In spite of the advantages in terms of efficiently serving the society, the risk of damage to pipelines is a key consideration from a sustainability point of view. Any significant disruption to the performance of pipeline systems often translates into undesirable impacts on regional businesses, economies, the living conditions of citizens, along with likely adverse effects on the environment (e.g., risk of harm to the environment from loss of integrity in oil pipelines). Based on available data from the U. S. Pipeline and Hazardous Materials Safety Administration (PMHSA 2011), the average damage costs arising from significant pipeline damage incidents over the past 10 years have been in excess of \$400M/year. Pipeline damage statistics from Canada, Europe, and the United States indicate that about one-eighth of all pipeline damage incidents over the last 30 years have been due to geotechnical hazards. It has been noted that pipeline failures induced due to ground movements would typically lead to damage costs more than double those arising from other hazards.

In addition to the loads from the transported content, it is critical that buried pipelines accommodate external loads that are transmitted through the surrounding soils (generally referred to as “soil loads”). The soil loads on pipelines can be wide ranging, and they include one or a combination of the following loadings that arise during different phases over the lifespan of a pipeline system: (i) pipeline installation; (ii) general operation; and (iii) extreme situations (e.g., landslides, earthquake-induced ground movements, etc.). Therefore, geotechnical engineering, the sub-discipline of civil engineering that addresses the concerns related to the use of earth as an engineering material, has a critical role to play in the design, construction, and satisfactory operation of pipelines.

This paper is aimed at presenting geotechnical considerations associated with pipeline engineering with particular reference to the above phases. Pipeline materials and installation methods are briefly discussed at the outset and the approaches available to identify and quantify geotechnical hazards on buried pipelines, and engineering solutions to mitigate geotechnical hazards are then discussed.

## 2. Pipeline materials and installation methods

### 2.1 Pipeline materials

The type of pipeline material selected is dependent on many factors. When transporting content under high pressure, the expected maximum internal pressure mostly governs the pipe design. Pipe

materials with good stress crack resistance, low permeation, high impact resistance, and satisfactory UV performance are often sought by the designers.

Welded large diameter steel pipes are often used in industry applications for conveying pressurized water, oil, and natural gas. Usually, the exterior of steel pipes are specially coated to protect against corrosion and abrasion. Due to the high strength and ductile nature, steel pipes are equally good in many applications involving rough terrains or regions where ground movements are expected. Polymeric plastic piping (e.g., PVC - Polyvinyl chloride, PE - Polyethylene, PB - Polybutylene, and PP - Polypropylene) is also used for industrial applications such as sewer and municipal and industrial waste, natural gas distribution, potable water transportation or as subsoil field drains. The plastic pipes are considered advantageous because of their lower material, installation, and maintenance costs and their high corrosion resistance and extensibility.

The pipes used for district heating have a composite cross-section since they are typically pre-insulated, bonded DH pipes where three different materials are combined in the manufacturing process. These materials include: steel pipes for the supply of hot water, surrounding insulation foam made of polyurethane, and an outer pipe coating made of polyethylene. Consequently, the interface between the pipe and the surrounding soil is polyethylene (Weidlich and Wijewickreme 2012).

## **2. 2 Pipeline installation methods**

Installation of the pipelines below the ground surface has been found to be very attractive since it provides a convenient mode of supporting and protecting the pipelines. The determination of critical loads for engineering design of buried pipes requires consideration of the internal pressure of transported fluid and/or external loads from the soil surrounding. For example, the design of high pressure lines under operating conditions is mostly governed by the internal content pressure (e.g., oil and gas pipelines) as the external soil loads are comparatively small under typical operating conditions. In comparison, for pipelines with relatively low internal pressure (e.g., water and sewer pipelines), the soil overburden loads can be a significant operating consideration.

Most pipelines are buried at shallow depths below the ground (less than 1.5 m) for the ease of installation and access during maintenance or repair. The open-cut (or cut-and-cover) is the most common method of pipe installation in such shallow burial depths, in which a trench is excavated from the ground surface, and then pipes are placed to meet a certain gradient and alignment (Figure 1). In the presence of space constraints, earth support systems of trenches and groundwater dewatering may be required. The trench is backfilled with select material types or with the native soil. When pipeline are below roadways, the backfill often requires to be compacted to a specified density.

Buried pipelines need to be located at greater depths in situations such as below-water river crossings (see Figure 2), mountain passes, etc., and trenchless techniques are adopted in such installations (Chehab and Moore 2008; Moore 2010). Some of the technologies for trenchless installations include: (a) horizontal auger boring; (b) horizontal directional drilling (HDD); (c) micro-tunneling; (d) pipe ramming; (e) moling; and (f) pipe bursting. Depending on the available or chosen alignment,

trenchless pipeline installation processes can be economically and technically demanding, however, the methods are becoming increasingly popular due to the advantages in protecting the environment and impacting developed areas. The North American Society for Trenchless Technology (NASTT 2012) serves as one of the leading engineering societies and trade associations promoting and educating with respect to the renewal and installation of buried infrastructure using trenchless technology and the practical, social and environmental benefits thereof.



Figure 1: Typical open cut installation of buried pipelines.

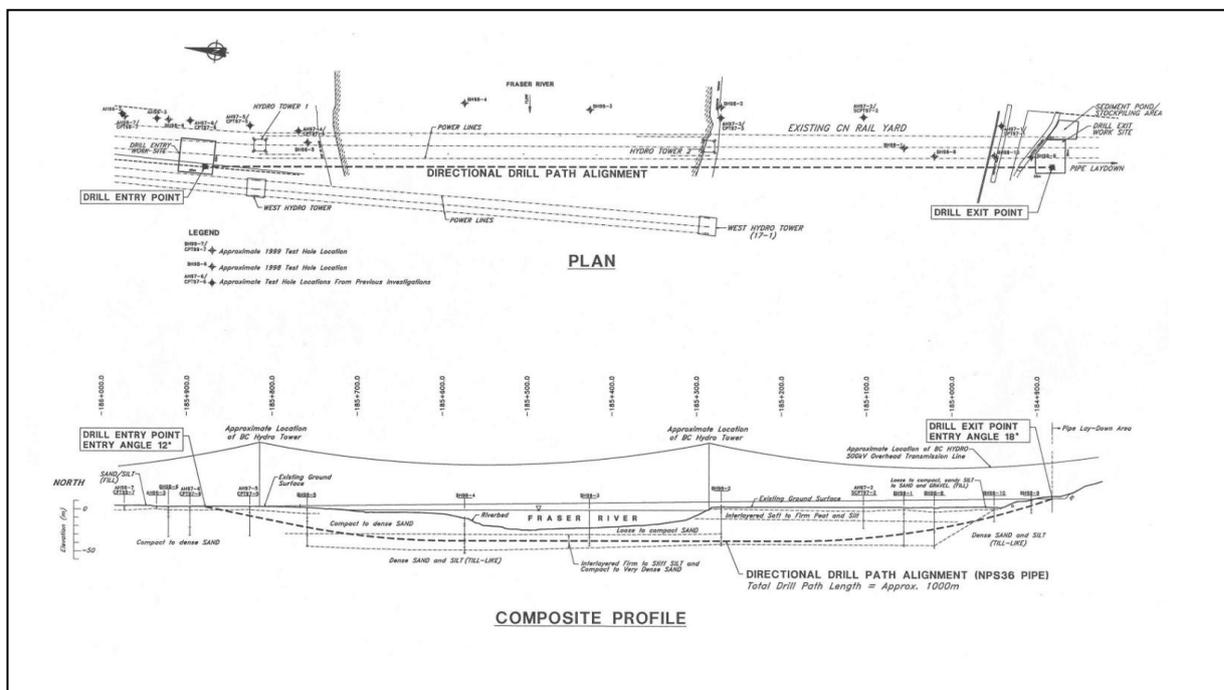


Figure 2: Example of a below-water pipeline river crossing; plan and profile of design pipeline alignment – FortisBC Fraser River Crossing Project, Vancouver, British Columbia, Canada.

### **3. General geotechnical engineering considerations**

Detailed understanding of subsurface soil and groundwater conditions along a given pipeline alignment through adequate geotechnical investigation is one of the keys to good design and construction of pipelines to meet the performance requirements. This is particularly important for trenchless techniques where special construction risks and significant challenges during pipe installation may arise due to potentially unforeseen soil and groundwater conditions.

Geotechnical investigations to determine site-specific soil and groundwater conditions could range from sampled test pits to other in-situ testing and geophysical profiling techniques. The extent of exploratory test holes (in terms of number and depth) and the investigation methodology should be based on the general knowledge of the regional geology, length/depth of the subject pipeline alignment, and the level of sophistication of the installation technique. Relatively deep drilled geotechnical test holes are typically required in cases with trenchless installations with deep alignments. While drilled test holes should be located close to the pipeline alignment, it is preferable that they are performed with at least a 10-m offset from the proposed alignment to minimize the risk of drill fluid escaping during trenchless pipeline installation work. If the initial findings indicate anomalies, discontinuity in the strata, the presence of rock or large concentrations of gravel, it is advisable to make additional borings to better define the strata. At below-water channel crossings, serious consideration should be given to carrying out adequate investigation within the channel section.

Laboratory sieve analyses and index testing are generally carried out on selected samples to classify the soils and to prepare detailed borehole logs, as appropriate. Information on the shear strength of fine-grained soils may be obtained using vane shear testing, unconfined laboratory compressive strength testing, etc. If rock is encountered, the type of rock and mineralogy, the Rock Quality Designation (RQD), the relative hardness, and the unconfined compressive strength should generally be determined.

### **4. Soil loads on pipelines during construction and general operational conditions**

Forces imparted by soils on a buried pipe is dependent on a number of factors: (i) the type and mechanical properties of the soil; (ii) the type and mechanical properties of the pipe material and pipe geometry; and (iii) pipe-soil interaction which is largely driven by the relative stiffness and strength of pipe materials in relation to the soil surrounding.

There have been many methods developed to determine the pipe performance under static loading. They range from closed-form solutions mainly developed assuming elastic pipe/soil material conditions; empirical and semi-empirical approaches; and numerical analyses methods based on two-dimensional and three-dimensional finite-element and finite-difference analyses supported by physical field and laboratory testing.

## 4.1 Static loads

Static soil loads on pipes buried in soils are based on the use of methods developed for determining geostatic stresses under free-field conditions (i.e., stress conditions prevalent without pipe). The vertical stress and horizontal stress, therefore, are expressed using the following formulae:

$$\sigma'_v = \gamma H \quad (1)$$

$$\sigma'_h = K \sigma'_v \quad (2)$$

where:  $\sigma'_v$  = vertical effective stress at a given depth;  $\sigma'_h$  = horizontal effective stress at that depth;  $\gamma'$  = average effective unit weight of soil above pipe;  $H$  = depth from ground surface to the point of interest;  $K$  = coefficient of lateral earth pressure (Note: under geostatic “at rest” conditions,  $K = K_0$  = coefficient of lateral earth pressure at rest). While the loads only from effective soil stresses are considered in the subsequent discussions, groundwater pressures should be accounted as appropriate. In the presence of groundwater, additional pore pressure could develop due to soil shear-induced volume change tendencies.

The weight of soil within the rectangular prism of soil having a width equal to the pipe diameter ( $D$ ) and a height equal to the depth of soil ( $h$ ) above the crown level of the pipe (termed "prism load",  $W_p$ ) is often used for computation of static soil stresses on pipes (see Figure 3). The “prism” load is generally used to determine the vertical load on relatively flexible pipes, except when such pipes are buried in micro-tunnels or pipes installed in jacked casings.

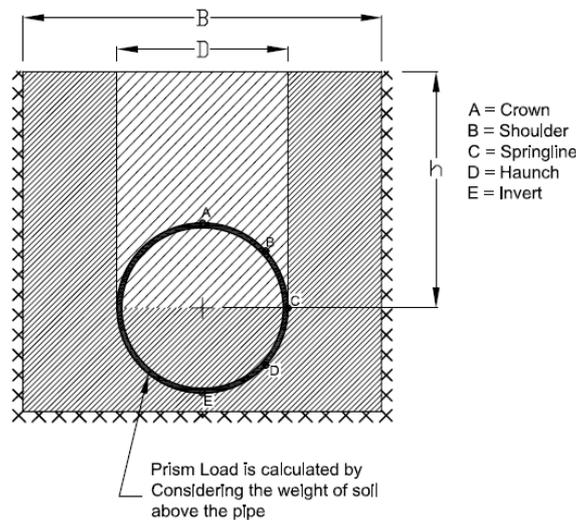


Figure 3: “Prism load” ( $W_p$ ) used in computation of static soil stresses on pipes.

Marston developed a theory to express the soil load on the pipe under this vertical stress. It is assumed that the settlement of the backfill and pipe generates shearing or friction forces at the sides of the

trench. With these assumptions, the soil load on a rigid pipe in trench installation can be obtained from the following:

$$W_d = C_d \gamma B_d^2 \quad (3)$$

$$C_d = [1 - \exp(-2K_\mu H / B_d)] / [2K_\mu]$$

$B_d$  = Trench width

$K_\mu$  = Soil friction coefficient (recommended value is 0.15).

$H$  = Depth of earth fill over pipe (in feet).

In situations with relatively rigid compacted soil embankments constructed over pipe alignments, or wide pipe trenches backfilled with compacted soil, negative arching can result in higher vertical stresses than the simple overburden stress in relatively rigid pipes.

## 4.2 Live loads

At pipeline crossings below roads, highways, or railways, live loads on the pipelines can be obtained from respective guidelines: e.g., ALA (2001) provides guidelines for the live loads based on HS20 of AASHTO (2002) for truck loads and Manual for Railway Engineering of American Railway Engineering Association (1975) for rail loads. Effects from point-live loads can be determined from Boussinesq's equation arising from elastic theory.

## 4.3 Buoyancy forces and thermal loads

The buoyancy force due to presence of water can become a significant factor for pipelines located below groundwater table, if the net upward buoyancy force exceeds the downward force arising due to earth loads and pipe weight. The most critical situation will arise when the pipe is empty or filled with air or gas. Buoyancy force per unit length of straight buried pipelines located entirely below the water table can be obtained as per guidelines by ALA (2001).

Pipelines may also be subjected to expansion or contraction due to temperature changes caused by seasonal temperature variations or by the temperature difference in soil and fluid conveyed through the pipeline. In the presence of end restraints from soil friction and/or by physical components such as pipe bends and joints, the pipes can develop compressive or tensile forces due to the thermal changes. The maximum stress arising due to thermal loading ( $\sigma_c$ ) can be conservatively estimated assuming fully restraint conditions at the pipe ends as suggested by the American Lifeline Alliance (ALA 2001). If the thermal loads are significant, pipes may be thermally insulated. If plastic pipe could be exposed to very high temperatures prior to insertion or direct burial, this initial temperature must be considered and the corresponding thermal loads should be calculated.

## 4.4 Deformations in low pressure pipes due to soil stresses

In pipes with low internal pressure subject to external static loading, a given pipeline can be categorized as relatively rigid or relatively flexible. In rigid pipes, pipe stiffness is significantly large compared to the stiffness of the soil, whereas, in flexible pipes, the in-plane pipe wall bending stiffness of the pipe can be comparable to or smaller than soil. The transmission of soil loads to the pipe is influenced by the relative stiffness of the pipe with respect to the soil – i.e., a soil-pipe interaction problem. For example, the soil-load carrying capacity of flexible pipes made of corrugated metal, thin-walled steel, or plastic is heavily dependent on soil-pipe interaction. Moreover, pipe bedding and other support conditions are critical to the ability of a flexible pipe system to withstand loads.

A given pipe, in the absence of lateral support, has only limited vertical load carrying capacity since the tendency would be for the pipe to deflect laterally under vertical loads. In most instances, lateral support derived from surrounding soil has an important role to play in maintaining the stability of buried pipes. It is also of interest to note that the ring deflection at the crown will induce the soil to arch, in turn, developing a tendency for relieving a significant portion of the vertical soil load. With proper understanding and accounting of these combined pipe-soil interaction effects in the vertical and horizontal directions, buried flexible pipe-soil system can be designed effectively to meet the performance criteria.

Spangler (1956) developed a method to calculate the change in horizontal diameter of flexible metal pipes once the total vertical load on the pipe per unit length have been determined. Similar methods have been developed over the years (e.g. Burns and Richard 1964; Hoeg 1968; Einstein and Schwartz 1979; and Moore 1987). The soil deformation modulus ( $E'$ ) represents the supporting strength of the surrounding soil which has considerable influence on the deflection of flexible pipes. Although the direct measurement of  $E'$  is not generally possible, this parameter is determined empirically based on the degree of soil compaction, soil type, effective stress conditions, etc. It is important to provide sufficiently rigid pipes in places where adequate lateral soil support cannot be assured.

## 5. Geotechnical hazards impacting pipelines

In addition to the loadings under operational conditions, potential loads on buried pipelines from external hazards are of importance in assessing the performance of pipeline systems since such loads can lead to potentially unacceptable strains in the pipelines. Typical hazards to pipelines include damage during construction, permanent ground deformation hazards such as slope failures (Figure 4), earthquake-induced soil displacements, ground movements at topographic discontinuities or faults, volcanic hazards, and tsunami inundation.

In particular, earthquake-induced permanent ground deformations have been recognized as one of the major causes of system damage and associated service disruption to lifeline facilities during past earthquakes (Hamada and O'Rourke 1992). The extent of permanent ground displacements is expected to increase with the increasing amplitude of earthquake accelerations and with the duration

of seismic shaking. These ground movement extents can be classified as flowslides (more than ~5m), lateral spreading flowslides (~5 m to ~0.3 m), and ground oscillation (less than ~0.3 m). In addition to liquefaction-induced ground movements, flotation and soil uplift could also be identified as another potential concern in relation to the reduction in soil strength associated with liquefaction. This would be of particular concern if the pipe trench backfill materials are poorly compacted and susceptible to liquefaction.

When it is necessary to assess wide-area pipeline networks, regional approaches are required to determine the ground displacement hazards (Wijewickreme et al. 2005). The approach proposed by Youd and Perkins (1987) provides a general method for mapping the liquefaction susceptibility based on the geological characteristics of a given area. Youd and Perkins (1987) have defined "liquefaction susceptibility" as the capacity of the soil to resist liquefaction (see Figure 5). They suggested that the age of the deposit, relative density, particle size, and depth to groundwater table are the primary factors that influence the liquefaction susceptibility. This approach provides a convenient and effective method of assessing the liquefaction susceptibility of a large area for a regional zonation study, where general surficial geological data are available but site specific data are limited.

The methods available for the computation of earthquake-induced permanent lateral ground displacements can be broadly classified into (a) empirical approaches developed based on measured displacements (e.g., Youd et al. 2002); and (b) mechanistic approaches which rely more on the principles of engineering mechanics (e.g., Newmark 1965; Byrne et al. 2004). Mechanistic methods mostly involve finite element or finite difference analyses, which are more appropriate for detailed site-specific analyses requiring a greater level of confidence. The estimation of earthquake-induced ground deformations, particularly from the viewpoint of regional assessments, still relies heavily on empirical correlations. Both empirical and mechanistic approaches would require the selection of earthquake magnitude and epicentral distance that is consistent with the probabilistic seismic hazard considered in the design.

Liquefaction can also produce overall volume changes in the liquefied soil mass that take place due to the dissipation of earthquake-shear-induced excess pore water pressures. The volume changes manifest in the field as post-liquefaction settlements, and they may occur both during and after earthquake shaking. The adverse impacts of these settlements on the performance of structural foundations and linear lifelines (such as buried pipelines and bridges) have been well recognized. Several simplified methods have been proposed to estimate settlements of soils knowing the field penetration resistance (i.e., standard penetration resistance N-value or cone penetration testing resistance) or laboratory cyclic shear resistance and the cyclic stress ratio corresponding to the level of ground shaking being considered (e.g., Tokimatsu and Seed 1987; Wu 2002; Wijewickreme and Sanin 2010). Much larger vertical movements are expected at river crossings, in the vicinity of dykes, ditches, road embankments, etc. due to distortion of the soil mass. Estimation of such vertical deformations would require rigorous site-specific analyses.

## 6. Pipeline response to geotechnical hazards

Evaluation of the performance of pipeline systems under anticipated geotechnical hazards commonly uses equations based on simplified assumptions or sophisticated numerical modeling techniques as described in guidelines dating back to mid-1970s (e.g., ASCE 1984) and recently updated (e.g., PRCI 2009).

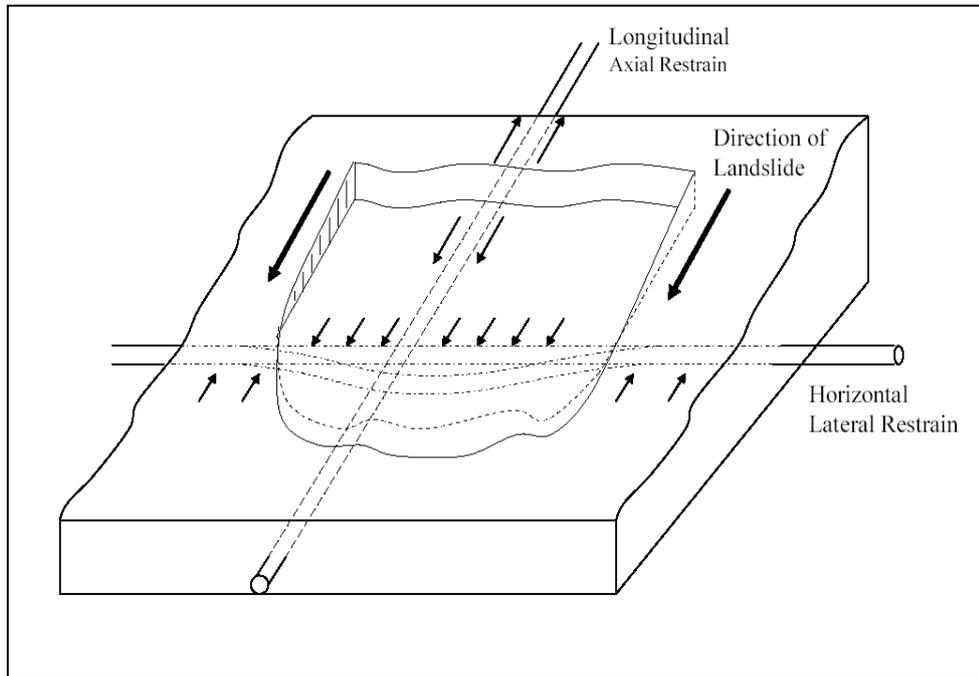


Figure 4: Soil loads on pipeline(s) during ground movements.

Nonlinear finite element techniques are the only practical means available to analyze all but the simplest problems. Methods relating to generic ground displacement values to the rates of pipeline damage, such as those incorporated into HAZUS risk assessment software (FEMA 2011) and presented in American Lifelines Alliance guidelines (ALA 2001) are usable for the assessment of non-welded pipelines with bell-and-spigot type connections; however, they are totally inappropriate for assessing oil and gas pipeline damage.

Finite element approaches provide a means to investigate the effects of changes in backfill characteristics, pipeline material, wall thickness, and pipeline alignment. The pipeline is modeled with pipe elements that are essentially beam elements that are capable of accounting for internal pressure effects. Soil restraint is modeled with non-linear spring elements that act independently in the axial, horizontal, and vertical directions relative to the axis of the pipeline (see Figure 6). Ground displacement induced by earthquakes is modeled as displacements applied to the base of the soil springs. There are no restrictions on the analysis software that can be used as long as it is capable of capturing the non-linear behaviour of soil springs, user-defined stress-strain curves for the pipe material, and large changes in pipeline geometry. The analysis software should include a pipe element

in its element library with the capability to model internal pressure and provide output at various circumferential locations.

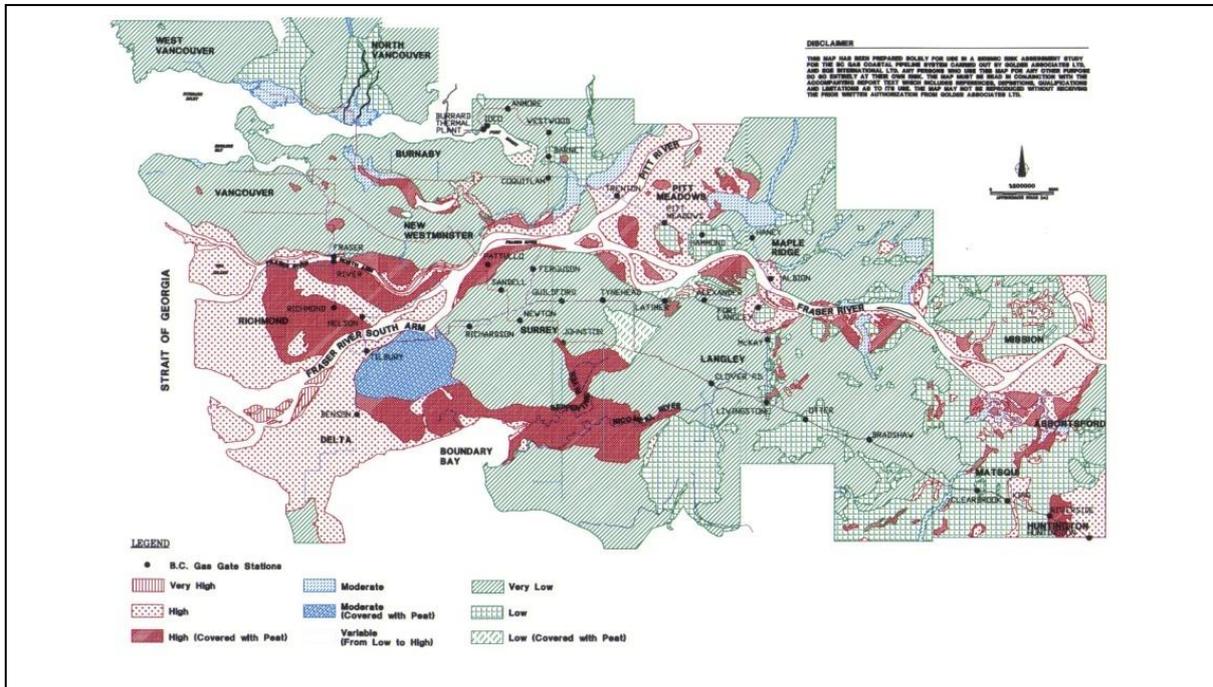


Figure 5: Liquefaction susceptibility map developed as a part of a pipeline study for FortisBC, Vancouver, British Columbia, Canada.

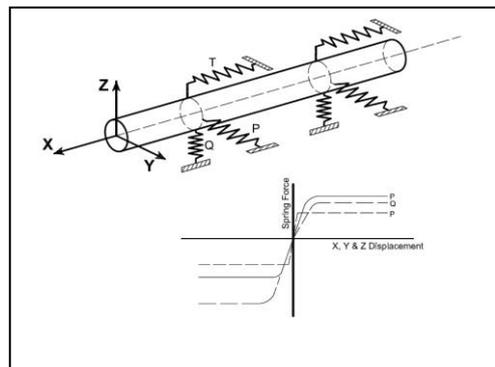


Figure 6: The use of soil-spring model for modeling for pipe-soil interaction at “element” level. (Note: Soil-springs: T = axial, P = horizontal, and Q = vertical directions relative to the pipeline).

The length of the pipeline model should be sufficient to adequately capture the anchoring effects of the soil outside the zone of ground movement. The pipe element length in regions where the pipe strain is expected to exceed the yield strain (typically at abrupt transitions in ground displacement or locations with abrupt changes in soil restraint such as elbows) generally should not exceed one pipe

diameter. The one-diameter rule is related to the fact that a gauge length of approximately one pipe diameter was the basis for reporting strains in tests used to establish strain acceptance criteria.

Available methods for modeling soil restraint with soil springs assumes that the spring forces always act in the axial, horizontal, and vertical directions relative to the pipeline. In most analyses, the direction of the soil spring forces do not maintain an axial, horizontal, and vertical orientation relative to the pipeline if the pipeline undergoes large rotations. The error introduced by this misalignment is acceptable considering other assumptions and uncertainties inherent in the analysis such as those related to the relationships used to compute soil spring properties. Most of this uncertainty is related to estimates of the soil strength parameters.

## **7. Mitigation methods to reduce risk to pipelines from geotechnical hazards**

Upon identification of the geotechnical hazards and the resulting vulnerability, a combination of structural retrofitting and/or geotechnical remediation (ground improvement) is often considered in the design of mitigation measures for buried pipelines against such hazards. Detailed site-specific studies are required to quantify potential for pipeline damage, and to determine whether or not practical alternatives exist to reduce the risk. Development of site-specific recommendations requires careful consideration of many factors including site geology, environmental conditions, pipeline response characteristics, and system performance requirements.

In general, there are four options to improve the performance of a given pipeline against an identified geotechnical hazard (i.e., ground movements): (a) avoid the hazard by relocation; (b) isolate the pipeline from the hazard; (c) accommodate the hazard by strengthening the pipeline or increasing flexibility; and (d) mitigate the hazard using ground improvement. Although avoiding the hazard by relocation is the most effective approach, this option is often not attractive because of prohibitive costs associated with acquisition of pipeline right-of-way for realignment.

The potential for pipeline failure can be reduced by reducing exposure of the pipeline to ground movement hazard. Other than rerouting, there are a few other methods to reduce pipeline exposure to the hazard. As indicated earlier, horizontal directional drilling techniques can be used to locate the pipeline below the zone of ground displacement. This technique is most commonly used to avoid lateral spread hazards at river crossings. Use of isolation culverts or above-ground supports provide effective means of isolating pipelines from ground movement hazards. The idea herein is to provide a mechanism for the ground that is subjected to movement to “slide past” the pipeline using a sliding support system. The above-ground isolation structure specifically designed to protect the Trans-Alaska Pipeline crossing of the Denali fault performed successfully during the 2002 M7.9 Denali fault earthquake, confirming the suitability of such isolation measures against geotechnical hazards. Moreover, soil restraint acting on the pipelines can be reduced by careful selection of pipeline trench geometry and backfill material, low-friction pipeline coatings, and wrapping pipeline with two layers of geotextile fabric, or placing a portion of the pipeline on the ground surface.

The potential for pipeline failure can also be reduced by increasing the pipeline wall thickness and material strength and also modifying the pipeline alignment to provide a more beneficial angle between the direction of ground displacement and the pipeline axis. The latter option is generally only possible during the design of new pipelines because of difficulties in obtaining new right-of-way.

Ground improvement is emerging as one of the widely adopted mitigation measures to reduce the impact of earthquake-related ground displacement hazards. In mitigation works, the design philosophy often revolves around implementing ground improvement measures to limit deformations in a given pipeline to acceptable levels (e.g., design to minimize the loss of pressure integrity in pipelines). Observations following major earthquake events have indicated that sites with improved ground had performed well during earthquakes (Mitchell et al. 1995). The ground improvement configurations used in practice are dependent on geotechnical risks that are to be mitigated, and these configurations essentially fall into one of the following categories: (i) in-ground densified barrier(s) aligned perpendicular to the direction of ground movement; and (ii) densification of wide-area footprints beneath and around foundation footprints.

The selection of the most suitable remedial option is governed by many factors including, but not limited to: soil conditions, space restrictions, issues related to the protection of existing structures during ground improvement, operational constraints, environmental regulatory requirements, and land availability. A variety of ground improvement techniques have evolved in the past few decades. These measures include dynamic deep compaction, vibro-replacement using stone columns, compaction piling, explosive compaction, and compaction grouting. The method of vibro-replacement using stone columns is the most preferred technique of ground improvement in sandy soils. The method can be effectively used to densify soils within about 25 m below existing ground level (see Figure 7). The method is attractive because of the potential availability of drainage through stone columns for the dissipation of excess pore water pressures in addition to the densification effect. Compaction grouting is a useful tool not only in fine-grained soils, but also in improving sites that have physical constraints such as low headroom. Deep dynamic compaction is another viable means of improving the settlement characteristics and liquefaction resistance of random fills and alluvial soils that are in a state of loose relative density and difficult for a probe to penetrate through. In-situ verification using penetration resistance measurements confirm that this method can be used to a maximum depth of about 10 to 12 m below existing ground level. Below this depth, the achieved improvement in penetration resistance diminishes considerably.

Verification testing for quality control forms a key component in undertaking ground improvement works. Evaluation of the treated soil type, method of ground improvement, and site constraints is required in selecting the parameters and testing tools to assess the conformance of ground improvement to specified criteria. In addition, effects from ageing and pore pressure dissipation can have significant influence on the observations from verification testing, and they should be carefully evaluated in determining the acceptability of a given ground improvement. The need to protect adjacent existing pipelines/structures is often a key consideration during ground improvement. Thus, systematic monitoring of existing facilities during ground improvement and structural evaluation of the performance of structures based on data from monitoring is essential.

## 8. Challenges and future trends

The available data on soil and groundwater conditions along pipeline alignments are typically limited. As a result, characterization of sites for soil-pipe interaction often becomes a challenging task. In addition, high variability of soil conditions along pipeline alignments combined with complexities associated with the mechanical behaviour of soil (e.g., stress and strain level dependence, effects of particle fabric, effects of loading paths, etc.) present additional constraints to the level of accuracy attainable in geotechnical evaluations. Sensitivity of the geotechnical hazard estimates to the above considerations along with the uncertainties arising due to the difficulties in accurately defining loadings such as earthquake shaking should be kept in mind when interpreting outcomes from hazard assessments.

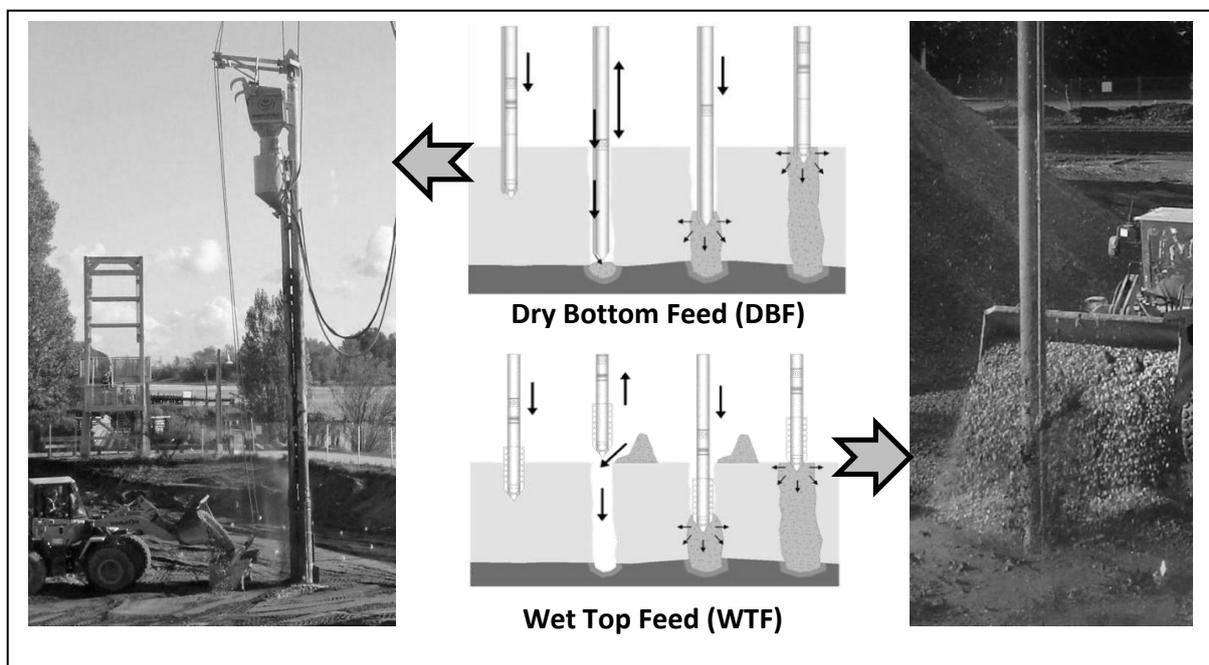


Figure 7: Illustration of the method of vibro-replacement methodology for densification of coarse-grained soils (Geopac Inc.) [Methodology: (i) Position vibro-probe vertically over compaction point; (ii) Penetrate to required target depth and terminate penetration; (iii) Begin feeding densification stone backfill from the skip bucket / vibro-probe hopper for DBF or directly at ground surface with wheel loader for WTF. (See WTF/DBF photos); (iv) Working in 0.5 m to 1.0 m depth increments starting from the maximum penetration depth; and (v) Continue building stone column at appropriate amperage level for each depth increment from maximum depth to the surface, while adding stone as required with wheel loader].

### 8.1 Numerical modeling

In general, soil-pipe interaction problems are statically indeterminate. The use of numerical modeling, in general, allows assessing the effect of a wide range of variables in a timely and efficient

manner. A vast amount of numerical analysis has also been performed to comprehend the complex interactions between pipe and soil (e.g., Rowe and Davis 1982; Guo and Stolle 2005).

The data generated from physical testing work would be of great applicability to calibrate and verify the numerical models. Despite the difficulties of attempting to use for routine engineering applications, continuum modeling methods have value in increasing the understanding of the general soil-pipe interaction problems, particularly with respect to improved definition of equivalent soil springs. One example in this regard is the need to understand the response of pipelines subject to ground movements that are oblique to the pipeline alignment; clearly, 3-dimensional continuum modeling becomes valuable in the modeling of such problems that cannot be captured in a 2-dimensional analysis. It also appears that discrete element methods seem to have a role to play in modeling complex soil-pipe interaction situations, especially when the particulate nature of soil seems to be the major governing factor of the observed response (e.g., modeling of the effectiveness of using geotextile interfaces, or coarse-grained trench backfills, in reducing soil loads on pipelines).

## **8.2 Non-linear pipe material behavior of plastic pipelines**

Polyethylene (PE) pipes (either high density polyethylene (HDPE) or medium density polyethylene (MDPE)) are becoming popular in natural gas distribution networks due to its lower material installation and maintenance costs, corrosion resistance, lower friction at the interface, lightweight, and its apparent capacity to accommodate larger displacements than steel pipes. The use of MDPE pipes takes advantage of having higher flexibility and fracture toughness while having comparable long-term strength and stiffness to that of HDPE (Stewart et al. 1999).

Reported research on the response of buried PE pipe systems subject to ground movement is very limited. Considering the relatively smaller deformation stiffness and time-dependent and nonlinear stress-strain response (viscoelastic and creep behavior) of PE pipe material, significant limitations could arise when methods developed for steel pipes are used for evaluating PE pipes. Data from controlled experimental work on pipelines subject to axial movement, particularly at full-scale level, is needed to advance the knowledge of the response of buried PE pipe systems subject to ground movement. A number of research programs have been already undertaken to investigate the response of PE pipe systems under permanent ground movements and analytical methods have been developed to account for the mobilization of soil loads in buried PE pipes under such ground movements (Weerasekara and Wijewickreme 2008).

## **8.3 Full-scale model testing**

Laboratory modeling of full-scale pipeline configurations is an attractive way of understanding the complexities in soil-pipe interaction. For example, full-scale testing provides a meaningful approach to estimate parameters of soil-springs to model the interaction between pipe and soil in different directions. Physical models simulating the field situations also play a key role in calibrating and validating the analytical approaches and numerical models. Some of the large soil chambers for full-scale testing of pipe-soil interaction problems are available at Cornell University (Trautmann and

O'Rourke 1983), Center for Cold Oceans Resources Engineering (Paulin et al. 1997), Queen's University (Moore and Brachman 1994), and University of British Columbia (Wijewickreme et al. 2009; see Figure 8 and 9 for details related to the Advanced Soil Pipe Interaction Research (ASPIRe™) test facility at the University of British Columbia (UBC)).

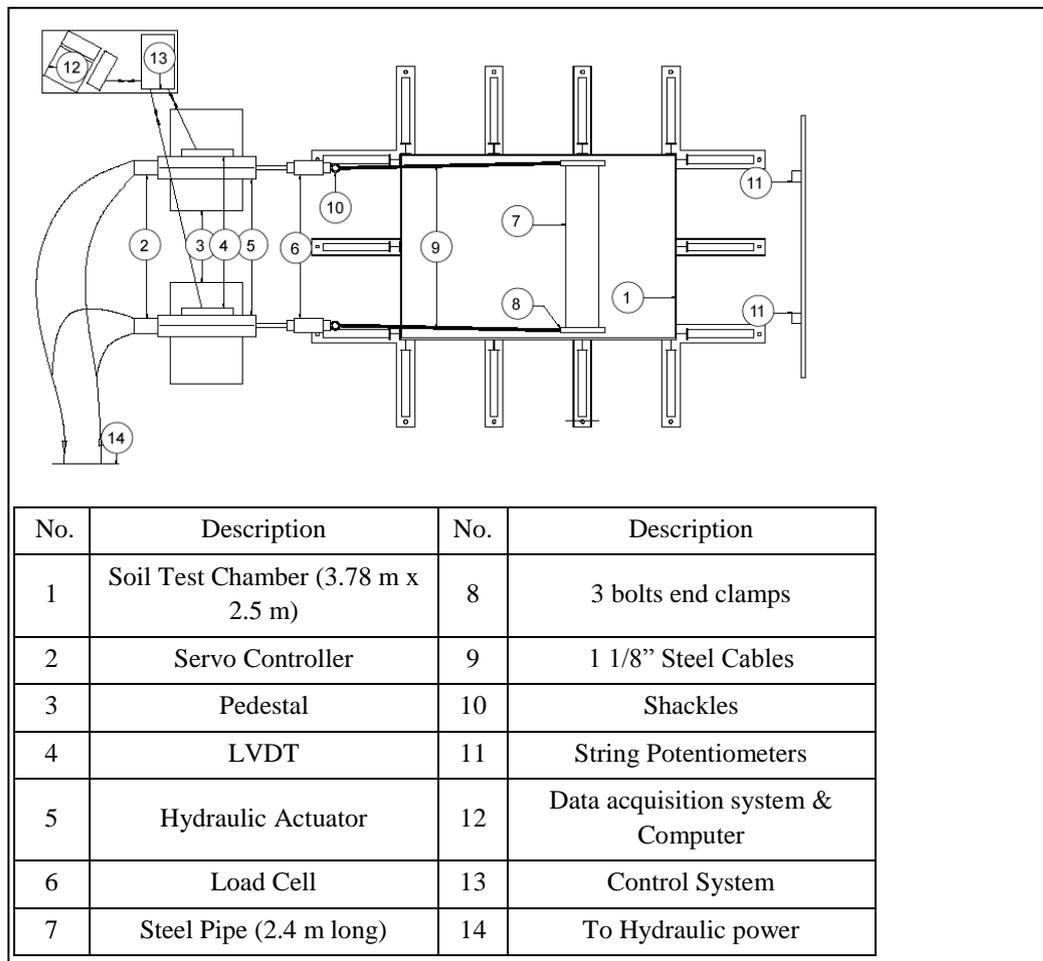


Figure 8: Layout of Physical Modeling Facility at UBC and Test System Components.

### 8.4 Field monitoring and testing of pipeline performance

Field monitoring of the pipelines and soil mass in the vicinity can play an important role with respect to several key aspects of pipeline geotechnical engineering. In particular, this allows: (i) assessment of imminent concerns; (ii) assessing pipeline performance in areas of varying, geotechnical conditions; and (iii) obtaining real-life data for validation of outcomes from numerical modeling of pipe-soil interaction problems.

Continuous field monitoring of pipelines become essential in situations where numerical modeling is complicated due to difficulties in determining the actual pipe-soil interaction because of field

variability on many fronts. An advantage in field monitoring is that they could be conducted over longer periods, without the space-time constraints that are common in laboratory environments.

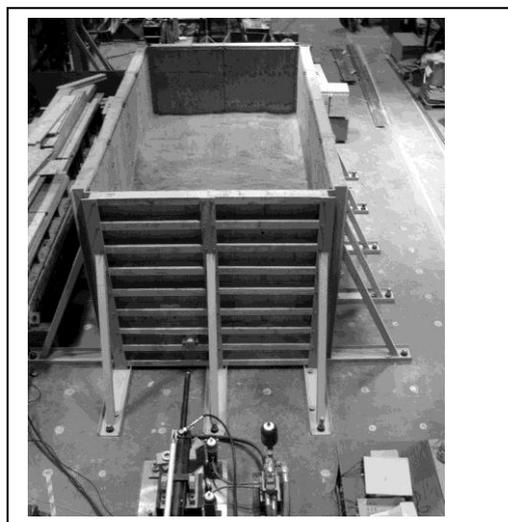


Figure 9: View of Physical Modeling Facility at UBC (set up for axial soil-pipe restraint testing).

## 9. Summary and Conclusions

Pipeline systems serve as a critical element in maintaining a sustainably built physical environment. Geotechnical engineering has a key role to play in ensuring satisfactory performance of buried pipelines in all of the following aspects of pipeline engineering: (i) construction/installation; (ii) routine operations; and (iii) extreme, less frequent, loading conditions (such as land-sliding, earthquake-induced ground movements, etc.).

The methods of installation of buried pipelines can be broadly classified into trenched (open-cut) installations and trenchless installations. Open-cut method is the simplest form of installation that involves placement pipe in an excavated trench and subsequent backfilling of the trench with soil. Trenchless technology uses horizontal drilling or jacking methods (i.e., horizontal auger boring, horizontal directional drilling, microtunneling, pipe ramming, moling, pipe bursting) where tools of varying levels of sophistication are used for alignment-tracking during installation work. Although generally more expensive than open-cut installations, trenchless methods are popular because of the advantages in protecting the environment and minimizing the impacts on developed urban areas.

Adequate knowledge of site-specific soil and groundwater conditions is critical to the success of a pipeline construction project, predicting the performance of a given pipeline system under operational conditions as well as during extreme loading situations. The soil loads imparted on pipelines can be wide ranging and typically include loadings that are encountered during day-to-day operations as well as those arising from extreme loadings that are less frequent. Estimation of pipeline performance is a difficult because of complex non-linear soil-pipe interaction. This task gets further complex due to

the difficulties in quantifying geotechnical hazards (e.g., landslide risk, liquefactions susceptibility and liquefaction-induced ground displacements, etc.).

In general, there are four options to improve the performance of a given pipeline against an identified geotechnical hazard: (a) avoid the hazard by relocation; (b) isolate the pipeline from the hazard; (c) accommodate the hazard by strengthening the pipeline or increasing flexibility; and (d) mitigate the hazard using ground improvement. In addition to mitigation, field monitoring and physical testing of pipeline performance are important considerations in the performance evaluation of buried pipelines.

## Acknowledgements

The research activities performed by Mr. Chris Anderson, Drs. Hamid Karimian and Lalinda Weerasekara as team members of the author's Advanced Soil Pipe Interaction Research (ASPIRE™) initiative are deeply appreciated.

## References

AASHTO (2002) *Standard Specifications for Highway Bridges* (Seventeenth Edition), Washington, DC, American Association for State Highway and Transportation Officials.

American Lifelines Alliance (ALA) (2001) Design of buried steel pipe, (available online [www.americanlifelinesalliance.com](http://www.americanlifelinesalliance.com) [accessed 28/12/2011]).

American Railway Engineering Association (1975) *Manual for Railway Engineering*.

ASCE (1984) *Guidelines for the seismic design of oil and gas pipeline systems*, American Society of Civil Engineers, Gas and Liquid Fuel Lifelines Committee, Technical Council on Lifeline Earthquake Engineering, pp. 157-170.

Burns J and Richard R (1964) "Attenuation of stresses around buried cylinders", Symposium on Soil-Structure Interaction. Tucson, AZ: American Society for Testing & Materials, West Conshohocken, PA, pp. 379-392.

Byrne P, Park S, Beaty M, Sharp M, Gonzalez L, and Abdoun T (2004) "Numerical modeling of liquefaction and comparison with centrifuge tests." *Canadian Geotechnical Journal* **41**(2):193-211.

Chehab A and Moore I (2008) "Axial force in pulled-into-place HDPE pipes during and after installation". *3rd Brazilian Conference on Trenchless Technology and 1st Latin America No-Dig*, Sao Paulo, Brazil, 10 pp.

Einstein H and Schwartz C (1979) "Simplified analysis for tunnel supports" *Journal of Geotechnical Engineering*, **105**(4): 499-518.

FEMA (2011) *HAZUS user manual*, Federal Emergency Management Agency. (available online [www.fema.gov/hazus/](http://www.fema.gov/hazus/) [accessed online 28/12/2011]).

Guo P and Stolle D (2005) “Lateral pipe-soil interaction in sand with reference to scale effect.” *Journal of Geotechnical and Geoenvironmental Engineering*, **131**(30): 338-349.

Hamada M and O’Rourke T, Eds. (1992) *Case studies of liquefaction and lifelines performance during past earthquakes: Japanese case studies*, Technical Report NCEER-92-0001, Vol. 1, Buffalo, NY: National Center for Earthquake Engineering Research, State University of New York.

Hoeg K (1968) “Stresses against underground structural cylinders.” *Journal of Soil Mechanics and Foundation Engineering*, **94**(4): 833-858.

Mitchell J, Baxter C, and Munson T (1995) “Performance of improved ground during earthquakes.” In RD Hryciw, Ed., *Soil Improvement for Earthquake Hazard Mitigation, Geotechnical Special Publication GSP No. 49, Proceedings of sessions, conjunction with the ASCE Conference*, San Diego, CA, 22-26 October 1995, New York, NY.

Moore I (1987) “The elastic stability of shallow buried tubes.” *Géotechnique* **37**(2):151-161.

Moore I (2010) “Design Tools for Directional Drilling”, *Association of Professional Engineers and Geoscientists of British Columbia, Annual Conference & AGM*, 21-23 October, 2010, Whistler, BC.

Moore I and Brachman R (1994) “Three dimensional analysis of flexible circular cylinders.” *Journal of Geotechnical Engineering*, **120**(10): 1829-1844.

North American Society for Trenchless Technology (NASTT) (2012), General website, (available online <http://www.nastt.org/> [accessed on 30/11/2012]).

Newmark N (1965) “Effect of earthquakes on Dams and embankments.” *Geotechnique* **15**: 139-160.

Paulin M, Phillips R, Clark J, Hurley S, and Trigg A (1997) ‘Establishment of a full-scale pipeline/soil interaction test facility and results from lateral and axial investigations in sand,’ *16th international conference of offshore mechanics and arctic engineering, OMAE 5*, American Society of Mechanical Engineers, pp. 139-146.

Pipeline Hazardous Material Safety Administration (PHMSA) (2011) (available online <http://www.phmsa.dot.gov> [accessed 28/12/2011]).

Pipeline Research Council International (PRCI) (2009) *Guidelines for constructing natural gas and liquid hydrocarbon pipelines in areas subject to landslide and subsidence hazards*, Catalog No. L52292.

Rowe R and Davis E (1982) "The behaviour of anchor plates in sand." *Geotechnique* **32**(1): 25-41.

Spangler M (1956) "Stresses in pressure pipe-lines and protective casing pipes". *Journal of Structural Engineering* (82): 1-33.

Stewart H, Bilgin O, O'Rourke T and Keeney T (1999) *Technical reference for improved design and construction to account for thermal loads in plastic gas pipelines*. Technical report, Ithaca, NY: Cornell University.

Tokimatsu, K and Seed, BH (1987) "Evaluation of settlements due to earthquake shaking." *Journal of Geotechnical Engineering*, **1138**(8): 861-879.

Trautmann C and O'Rourke T (1983) *Behaviour of Pipe in dry sand under lateral and uplift loading*. Geotechnical Engineering Report, Ithaca, NY, Cornell University.

Weidlich I and Wijewickreme D (2012) "Factors Influencing Soil Friction Forces on Buried Pipes Used for District Heating", *13th International Symposium on District Heating and Cooling*, Copenhagen, Denmark, 2-4 September, 2012.

Weerasekara L and Wijewickreme D (2008) "Mobilization of soil loads on buried polyethylene natural gas pipelines subject to relative axial displacements." *Canadian Geotechnical Journal*, **45**(9): 1237-1249.

Wijewickreme D, Honegger D, Mitchell A, and Fitzell T (2005) "Seismic vulnerability assessment and retrofit of a major natural gas transmission system: A case history." *Earthquake Spectra*, **21**(2): 539-567.

Wijewickreme D, Karimian H and Honegger D (2009) "Response of buried steel pipelines subjected to relative axial soil movement." *Canadian Geotechnical Journal*, **46**(7): 735-752.

Wijewickreme D, and Sanin M (2010) "Post-cyclic reconsolidation strains in low-plastic Fraser River silt due to dissipation of excess pore water pressures." *Journal of Geotechnical and Geoenvironmental Engineering* **136**(10): 1347-1357.

Wu, J (2002) *Liquefaction triggering and post-liquefaction deformation of Monterey 0/30 sand under unidirectional cyclic simple shear loading*, Ph.D. thesis, University of California, Berkeley, CA.

Youd L and Perkins D (1987) "Mapping liquefiable ground failure potential." *Journal of the Geotechnical Engineering Division*, American Society of Civil Engineers, **104**(4): 433-445.

Youd T, Hansen C, and Bartlett S (2002) "Revised multilinear regression equations for prediction of lateral spread displacement." *Journal of Geotechnical and Geoenvironmental Engineering*, **128**(12): 1007-1017.