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Seismic risk assessment for oil and gas pipelines

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Abstract: Buried pipeline systems form a key part of global lifeline infrastructure, and any significant disruption to the performance of these systems often translates into undesirable impacts on regional businesses, economies, or the living conditions of citizens. This chapter addresses the considerations associated with the seismic risk assessment of pipelines providing transmission of natural gas or liquid hydrocarbons, and pipelines that are part of a gas distribution system serving a region. Particular reference is made with respect to philosophy, approaches, and technologies adopted in designing and operating pipelines to minimize pipeline damage during earthquakes.

Key words: pipeline, risk, soil–pipe interaction, performance goal.

25.1 Introduction

Quantification of anticipated seismic hazards is a key consideration in assessing performance of pipelines under seismic loading conditions. Evaluation of the performance of pipeline systems under such hazards commonly uses equations based on simplified assumptions or sophisticated numerical modeling techniques as described in guidelines dating back to the early 1980s (e.g., ASCE, 1984) and more recently updated (e.g., PRCI, 2009).

This chapter addresses the considerations associated with the seismic risk assessment of pipelines providing point-to-point transmission of natural gas or liquid hydrocarbons, and major distribution pipelines that are part of a natural gas distribution system serving a region or local community. Liquid hydrocarbons include a variety of products: crude oil, refined products such as gasoline, diesel fuel, jet fuel, lubricating oil, and gas liquids such as propane. For simplicity, hydrocarbon pipelines will be referred to as 'oil' pipelines in this discussion. The scope of this discussion does not include pipelines within facilities such as onshore drilling fields, offshore platforms, refineries, or major distribution facilities. The scope also does not include smaller pipelines incorporated into a municipal distribution system, which normally operate at lower pressures and are constructed of materials other than steel. This scope is similar to that defined for liquid hydrocarbon and other pipelines in ASME B31.8 (ASME, 2007), with the only difference being the extension to the first pipeline connection within a facility in the scope of ASME B31.8.

25.2 Purpose of performing a risk assessment

Risk is formally defined as the product of the probability of an event occurring and the consequences of the event. This definition implies that risk be stated in a quantitative manner. For example, a 10% chance per year of an event causing the death of one person has a risk of 0.1 deaths per year. In the context of this chapter, a distinction is made between risk assessment and risk management or, as commonly referred to in the oil and gas pipeline industry, integrity management. The term risk assessment is appropriate for quantitative estimates of both the potential for damage and potential consequences. The basis for determining the quantitative value of risk is typically highly dependent upon qualitative descriptions that can be translated into numerical values. For example, the likelihood of earthquake-triggered slope movement sufficient to cause pipeline failure given a peak ground acceleration of 0.4*g* might be ranked as low, moderate, or high and subsequently assigned numerical values of 5%, 50%, and 85% for use in a risk calculation.

The goal of any risk assessment is to provide information to answer some specific questions that necessarily includes a scenario, a performance metric, and an acceptance criterion. Examples of questions with reliability scenarios include the following:

- What is the annual probability that earthquake damage (scenario) will interrupt service to customer XYZ (performance metric) for more than 24 hours (acceptance criterion)?
- What is the probability that costs (performance metric) to repair earthquake damage (scenario) will exceed $2 000 000 over the next 15 years (acceptance criterion)?

Other categories of system performance metrics and acceptance criteria are provided in Table 25.1.

As of 2012, there are very few regulations for oil and gas pipelines that explicitly define risk acceptance criteria and require a quantitative seismic risk assessment to demonstrate compliance. Integrity management regulations in North America (PHMSA, 2011) are directed at long-term measurements of performance and gradual system upgrades and do not specify specific performance requirements. Some local agencies in the United States rely upon risk assessments for land use planning (e.g., County of Santa Barbara, 2000). Annex O of the Canadian pipeline code (CSA, 2007)

Table 25.1 System performance metrics and acceptance criteria

	System performance metrics					
Acceptance criterion	Capital loss ($)	Revenue loss ($)	Service interruption (% customers)	Downtime (hrs, days)	Casualties (injuries, deaths)	Lost product (volume)
Number of injuries or deaths					X	X
Duration of service interruption			X			
Number of customers served				X		
Amount of monetary loss	X	X	X	X		X
Quantity of release						X

provides an optional method that relies upon risk assessment to demonstrate suggested reliability targets have been met. More explicit reliance on risk assessments as a tool for gauging regulatory compliance are largely confined to government agencies, most prominently those agencies responsible for deciding the feasibility of recalls in the automotive and aviation sectors to correct potential safety issues. Outside the regulatory environment, the relevance of risk assessments for oil and gas pipelines is largely delegated to academic studies focused on regional or national economic and societal impacts of extreme events (e.g., O'Rourke *et al.*, 1992) and targeted studies performed for the insurance industry.

Information from the risk assessment can be used to gauge regulatory compliance, when regulations dictate specific reliability targets, but are most often used to support cost versus benefit decisions on the part of regulatory agencies or the pipeline owner. It is more common for oil and gas pipeline owners to rely upon vulnerability assessment in which some quantitative measure of the likelihood for pipeline damage is established. The information on vulnerability is primarily used to guide decisions on capital improvement projects related to increasing resiliency through pipeline replacement or redundant transmission paths. In this approach, utility personnel are responsible for relating the vulnerability to consequences, often based upon qualitative or ranking measures, to determine whether identified risks are unacceptable.

25.3 Key steps in performing risk assessments for oil and gas pipelines

Risk assessment of oil and gas pipelines involves many uncertainties. Understanding these uncertainties and their causes is required to interpret the results of the risk analysis. The analysis of uncertainties associated with data, methods, and models used to estimate the risks involved should play an important part in the overall risk analysis process.

The first step is to define the scope of the risk analysis. This involves defining the objective of the risk analysis and includes consideration of the configuration of the system being assessed, the physical and functional boundaries of the system, the environment in which the system operates, operating conditions for the system, and the technical, environmental, organizational, and human circumstances relevant to the system. It is also necessary to determine whether a qualitative or quantitative method of risk assessment is appropriate for the scope and goals of the risk analysis.

It is important to clearly define what constitutes a tolerable level of risk at the outset of the process. Defining tolerable exposure criteria early on in a site-specific risk analysis can greatly simplify the effort involved in the risk analysis as some events may be able to be quickly identified as having no significant consequence without detailed calculation. Unfortunately, defining tolerable risk is often complicated by emotional issues related to explicitly defining an acceptable likelihood for injuries or deaths. Examples of acceptable risk levels adopted by various industries and organizations are provided in Table 25.2.

The second step is to define the potential hazards to pipelines in question and determine the likelihood of those hazards occurring. These are expected to include pipe damage from ground displacement (e.g., landslides, liquefaction, and settlement), pipe damage from third party activity, and pipe damage related to typical operational conditions (e.g., Kiefner and Trench, 2001). In many countries, regulators require oil and gas pipeline operators to report significant pipeline accidents and this information can provide the basis for estimating the likelihood of hazard occurrence. In some cases,

Table 25.2 Examples of acceptable annual probabilities

Reference	Annual exceedance probability (AEP)	1/AEP
Need for accident planning unwarranted or unnecessary except for catastrophic accident[1]	0.001000	1 000
Point for diminishing justification for reducing US Bureau of Reclamation dam failure[2]	0.000100	10 000
California gas transmission pipeline incident rate (1984–2001)[3]	0.000074	13 513
Building collapse from seismic load (US)[4]	0.000040	24 750
Highest safety class for nuclear processing facilities[5]	0.000010	100 000
DNV OS-F101 lowest failure probability for high risk of human injury[6]		
Equivalent annual probability of earth impact by asteroid Apophis in 2036[7]	0.000004	250 000
Lower range of probability for core damage to existing nuclear power plants[8]	0.000002	500 000
Estimate of lower range of probability for core damage for proposed new nuclear power plants[8]	0.000001	1 000 000
Acceptable individual risk for student established by California Department of Education[3]		

[1] FEMA/DOT/EPA (1988).
[2] US Department of Interior (1997).
[3] California Department of Education (2007).
[4] ASCE 2010 (assumes Risk Categorgy 2 and annual probability of MCE of 1/2 475).
[5] Kennedy (1992).
[6] Det Norske Veritas (2000).
[7] NASA (2009).
[8] Nuclear Energy Institute (2012).

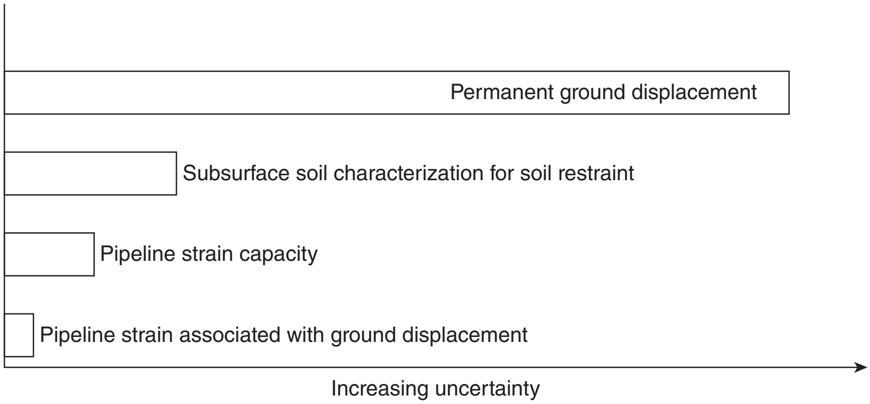
these hazards are well known. In other cases, hazards can be identified through formal methods that include hazard and operability studies, failure mode and effect analysis, fault tree analysis, and event tree analysis. Each of these methods has advantages and disadvantages associated with the ability to identify all relevant hazards, the level of complexity, and the necessary input data requirements. In many cases, a simple method that is well performed and meets the objectives and scope of the risk analysis can provide better results than that from a more sophisticated analysis. Ordinarily, the effort put into the risk analysis should be consistent with the potential level of risk being assessed. For seismic risk assessment, earthquake hazards are directly linked to a particular earthquake size, location, and style of faulting. They are either a direct hazard such as ground shaking and surface fault displacement or an indirect hazard such as triggered slope movement, liquefaction, lateral spread displacement, and post-cyclic consolidation settlement.

The third step is to estimate the level of pipeline vulnerability for each hazard event based upon an analytical assessment of the amount of stress or strain developed in the pipeline. A distinction is typically made between modest damage modes, such as leaks or holes in the pipe wall, and more catastrophic damage modes, such as a full line break. Determining the potential for unacceptable pipeline performance requires relating the likelihood of an earthquake to the severity of the earthquake hazard and then relating the severity of the earthquake hazard to pipeline response. There are several sources of significant uncertainty in determining the potential for earthquake induced pipeline damage:

- estimated earthquake-induced ground displacements which are related to:
  - earthquake recurrence rate,
  - earthquake location,
  - triggering of indirect earthquake hazards;
- subsurface soil characteristics for determining soil restraint on buried pipelines;
- pipeline strains produced by ground displacements;
- pipeline strain capacity for a particular performance level (e.g., continued operation and pressure integrity).

A qualitative comparison of the relative uncertainty in each of these parameters is illustrated in Fig. 25.1.

The fourth step in the risk assessment process involves estimating the likelihood of various consequences of pipeline damage. For natural gas pipelines, the effects of gas release typically include toxicity of released materials, such as hydrogen sulfide, thermal radiation due to ignited gas jets emanating from small holes or tears in the pipe wall, and external pressure



25.1 Qualitative depiction of relative uncertainty in key vulnerability assessment parameters.

and thermal exposure from the ignition of a gas cloud. For oil pipelines, the effects of oil release typically include thermal radiation from pool fires, property damage from inundation with oil, and environmental contamination. The assessment of consequences involves many uncertain parameters, such as those listed below:

- size of the pipe opening;
- location of the pipe opening around the pipe circumference;
- availability of ignition source;
- duration of leakage;
- vapor cloud dispersion parameters (e.g., wind speed, wind direction, and terrain roughness);
- number and location of persons near the pipe opening;
- number and location of structures near the pipe opening.

The measure of risk is defined both in terms of vulnerability and consequences (e.g., 0.01% chance per year for a fatality from thermal radiation). This risk measure is reviewed for acceptability and to identify possible actions to reduce unacceptable risk. This is normally an iterative process as the potential reduction in risk is measured by re-evaluating changes produced by the identified actions on the likelihood of a hazard occurrence, the consequences of the hazard, and combined risk.

25.4 Types of seismic hazard

In addition to the loadings under operational conditions, potential loads on buried pipelines from seismically induced hazards are of importance to assessing the performance of pipeline systems.

The typically seismic hazards to pipelines include:

- wave propagation;
- permanent ground deformation hazards:
  - differential movements at topographic discontinuities or faults,
  - slope failures,
  - liquefaction-induced permanent ground deformations;
- volcanic hazards;
- tsunami inundation.

25.4.1 Wave propagation

Seismic wave propagation is a ground motion phenomenon that relates to the passage of body waves, including compression waves and shear waves, radially from the source of earthquake energy release (hypocenter) into the surrounding rock and soil medium. Compression waves cause compressive and tensile strains in the ground in a radial direction away from the hypocenter. Shear waves cause shear strains in the ground perpendicular to these radial lines. When the compression waves and shear waves are reflected by interaction with the ground surface, surface waves (Love waves and Rayleigh waves) are generated. Except at large distances from the epicenter, the amplitudes of surface waves are much less than body waves. An earthquake at its source generates only compression and shear waves, and propagation of its radiated waves can be evaluated using ray theory (Pujol, 2003). Since the amplitude of shear waves is significantly larger than compression waves and thus generates greater strains in a pipeline, the examination of wave propagation can be limited to the effects of shear waves. Also, the strongest component of ground shaking in strong motion instrument records used to derive attenuation relationships is typically from shear waves (Bolt and Abrahamson, 2003).

A pipeline buried in soil that is subject to the passage of these seismic waves will incur longitudinal and bending strains as it conforms to the associated ground strains. In most cases, these strains are relatively small, and welded pipelines in good condition typically do not incur damage. Propagating seismic waves also give rise to hoop membrane strains and shearing strains in buried pipelines, but these strains are small and may be neglected.

25.4.2 Permanent ground deformation

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). Essentially, large soil loads arising from permanent ground movements can lead to potentially unacceptable strains in the pipelines. Extensive research has been focused in the last few decades to investigate the response of the pipes under permanent ground deformations. Common causes of permanent ground displacement are related to surface fault displacement, liquefaction-induced lateral spreading and flow slides, slope instability and landslides, and ground subsidence.

Ground movements at fault crossings

Pipelines crossing faults can be subjected to displacements ranging from a few centimeters to several meters. In addition to the lateral movements significant vertical movements can occur in the crossings involving reverse-thrust faults. Large ground movements at the Trans-Alaska Pipeline crossing of the Denali fault near Glennallen, Alaska, during the 2002 M7.9 Denali fault earthquake is a classic example of the type of ground movements expected at fault crossings.

Liquefaction-induced ground movements

In saturated loose or soft granular soils, liquefaction occurs when the shear strains induced due to seismic shaking cause transient pore water pressures to increase in the soil mass. As a result, intergranular contact stresses will reduce to negligible levels. In this transient state, the soil mass is subject to significant reduction in shear strength and behaves essentially as a viscous fluid that could deform or flow under gravitational or inertia forces. Areas susceptible to liquefaction could undergo significant vertical and lateral movements even in gently sloping terrain. The extent of permanent ground displacements is expected to increase with the increasing amplitude of earthquake accelerations and with the duration of seismic shaking. The extent of ground movements can be classified as: flowslides (more than ~5 m), 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.

Liquefaction can 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 (Tokimatsu and Seed, 1987; Wijewickreme and Sanin, 2010).

Ground movements due to landslides

Significant ground movements could occur due to landslides triggered (without soil liquefaction) during earthquake shaking. This would become a concern in areas of steep terrain and saturated slopes with soft soils, or in areas where there is ongoing relatively slow moving landslide activity.

25.4.3 Volcanic hazards

In terms of volcanic hazards, exposure to the hazard is generally related to the type and nature of the volcano (e.g. impact of volcanic hazards tends to be greater for andesitic volcanoes than for basaltic volcanoes) and to the proximity to the volcano edifice, as well as to whether it is on a drainage that emanates from the edifice. The potential volcanic hazards that may be present along the pipeline routes include tephra (ash) fall, pyroclastic flow/surge, blast surge, lava flow, mud flow, debris flow, ground deformations, and volcanic earthquakes.

25.5 Determining hazard likelihood

To be technically complete, seismic risk assessment should consider the integrated likelihood of exceeding a specific acceptance criterion from the total range of earthquake hazards. Such an approach accounts for the cumulative likelihood associated with a large number of frequently occurring earthquakes with a small probability of producing a vulnerability. In practice, the process is often simplified by assuming a single earthquake frequency that is acceptable for a particular level of vulnerability. For example, if an acceptable level of risk an annual probability that the pipeline will lose pressure integrity less than 0.5%, it is common practice to only address earthquake hazards that have an annual chance of being exceeded of 0.5%. This simplification allows the results of the risk assessment to be viewed in the context of other components such as buildings, dams, and bridges that are typically designed and evaluated for a single level of earthquake hazard.

The severity of earthquake ground shaking is commonly defined using probabilistic methods that include consideration of the variability in the size, recurrence interval, and location of earthquakes within a region. The other induced seismic hazards are typically defined based upon deterministic or empirical methods that may rely upon the probabilistic ground shaking estimate as an input parameter. It needs to be recognized that there is considerable uncertainty in any seismic hazard definition.

The range of uncertainty can be gauged by examining the typical uncertainty in estimating ground shaking. Considering only the uncertainty in attenuation and adopting the range of standard deviations in ground motion estimates presented in Lee *et al.* (2000), for a predicted peak ground acceleration value of 0.5 g, the actual peak ground acceleration would lie within 0.4 g to 1.0 g, 50% of the time. The actual variation would be larger if variability other than that associated with attenuation were considered. From this simple example, it should be clear that defining levels of ground shaking to the nearest 0.05 g is adequate for most risk assessment applications. Similarly, ground displacement hazards should not be defined with artificial precision.

25.6 Determining severity of hazard

The assessment of site-specific geotechnical hazards forms an important part in overall vulnerability assessment of a pipeline system and facilitates the development of potential mitigation measures. Permanent ground deformations triggered by earthquakes have been recognized as one of the major causes of system damage and associated service disruption to lifeline facilities. For example, significantly large permanent lateral deformations are expected to occur even under gently sloping ground conditions in areas of liquefiable soil. Therefore, an estimation of the extent of such ground displacements is important in the assessment of pipeline system vulnerability. In particular, adequate knowledge of site-specific soil and groundwater conditions is critical to the success of the design and installation of pipelines, as well as in predicting its anticipated performance under field conditions.

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., 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. It is important to note that both empirical and mechanistic approaches (or any combined approaches thereof) would require the selection of earthquake magnitude and epicentral distance that is consistent with the probabilistic seismic hazard considered in the design.

Except under high ground shaking intensity levels (i.e. in excess of peak ground acceleration of about 0.4 g), propagation of seismic waves through soft soils generally causes the ground motions to amplify as they reach the ground surface. This aspect is important and should be accounted in the liquefaction assessment and estimation of ground deformations in soft/loose soil zones.

25.6.1 Wave propagation strain

Pioneering work by Newmark (1967) still serves as the basis for estimating strains in pipelines arising from wave propagation. The Newmark approach has been incorporated, with minor modifications, into many guideline documents (ASCE, 1984; ALA, 2001a; Honegger and Nyman, 2004). The basic formulation of the Newmark approach for shear waves is provided in Equation 25.1:

$$\varepsilon = \frac{V}{\alpha C} \qquad [25.1]$$

where

ε = axial pipe strain,
V = peak horizontal ground velocity,
C = apparent shear wave propagation velocity (inverse of slowness between earthquake source and site),
α = coefficient dependent upon the incidence of the wave with the pipeline (2 for strain in a pipeline from shear waves, 1 for ground strain).

The inverse of the apparent propagation velocity ($1/C$ in Equation 25.1) is termed 'slowness' in seismology and is often noted by the variable 'p'. Tables of slowness are available in regions where detailed earthquake location studies have been undertaken and the value generally ranges from 0.2 s/km to 0.5 s/km. If such tables are not available, it is conservative to adopt the propagation velocity in bedrock at depth for the apparent propagation velocity. More detailed discussion of the phenomenon of wave propagation is provided in Litehiser *et al.* (1987). Litehiser *et al.* (1987) note two important findings with respect to the application of Equation 25.1. There is a potential for amplification of strain associated with surface soil layers compared with rock of approximately 1.5 to 2. There is also the possibility of further increased amplification, on the order of two times as great as the above, when the shear response of the surface soil layers is closer to being linear (i.e., for small ground motions) as opposed to an amplification on order of 1.5 when the soil response is nonlinear (i.e., for large ground motions).

The two findings from Litehiser *et al.* (1987) are consistent with Paolucci and Smerzini (2008) who determined variations of maximum ground strain for weak to moderate ground motions (most of the data from sites with peak horizontal ground velocities far less than 30 cm/s and peak ground accelerations less than 0.4 g) that were two to three times greater than what would be obtained assuming an inverse slowness of 2000 m/s.

Based upon Equation 25.1, it is clear that wave propagation strains are exceedingly small. Considering the largest ground velocities recorded are less than about 1.5 m/s and the lower bound value for the inverse of the slowness is 2000 m/s, and an amplification factor of 1.5 for nonlinear effects, an upper limit on the strain that can be generated from wave propagation is 0.056%. With the exception of early oxyacetylene welded pipelines and some pre-1945 pipelines fabricated with early arc welding processes (Honegger, 1999), oil and gas pipelines should be capable of tolerating longitudinal strains of 0.5%, the nominal yield strain for API 5L pipe. Assuming a longitudinal strain of 0.18% from internal pressure, the remaining longitudinal strain capacity available for oil and gas pipelines to accommodate wave propagation is nearly six times greater than what is expected to be typically generated from earthquake shaking. For this reason, wave propagation is an insignificant hazard to modern welded steel oil and gas pipelines and only need be considered for older pipelines when the axial strain capacity of the older pipelines is less than about 0.25%.

25.6.2 Site-specific estimation of permanent ground deformations

In general, for a given site, the site-specific ground displacements are estimated based on the following key steps: (a) geotechnical investigation to understand the site-specific soil and groundwater conditions; (b) assessment of site-specific loading parameters (e.g., changes to groundwater level under different weather conditions, and ground shaking intensity due to an earthquake) and ground response for the identified loading levels; (c) assessment of the geotechnical stability of the site; and (d) assessment of ground displacement hazard using empirical and/or mechanistic approaches (e.g., seismic slope stability, liquefaction-induced ground movements, and post-earthquake bearing capacity).

The deformation modulus and shear strength of liquefied soil are the key factors determining the extent of ground displacements. These parameters are often necessary for the assessment of seismic vulnerability and design of mitigation measures. Numerical approaches based on soil mechanics principles provide a means to estimate displacement patterns, but they often suffer from a lack of rigorous calibration with actual earthquake data.

25.6.3 Regional assessments of permanent ground deformations

When it is necessary to assess the seismic vulnerability of a wide-area pipeline networks, regional approaches are required to determine the ground displacement hazards. 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. 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 empirical methods have been mainly developed based on statistical/regression analyses of seismic, topographical, soils, and geological data associated with lateral spreads resulting from major earthquakes. Empirical methods are also unable to estimate ground displacement patterns on the surface and at different depths; therefore, they cannot account for the presence of man-made site features (i.e., the effects arising from zones of ground improvement). While the empirical models provide a convenient method to estimate liquefaction-induced ground displacements, use of the results should be made with a proper understanding of the limitations of the model, as well as the uncertainties in the input parameters and the model. Although such an approach is considered reasonable for regional pipeline study, more detailed site-specific analyses should be considered for locally identified risk areas. Moreover, even with the advantage of being based on an actual database of ground displacements while accounting for many physical parameters governing the 'free-field' ground displacements, empirical methods do not incorporate the deformation modulus and shear strength of liquefied soil in the computations.

25.6.4 Ground displacements due to landslides

In the case of site-specific assessments, the estimation of ground displacements due to earthquake-induced landslides (without liquefaction) is often undertaken using limit equilibrium-based stability analysis followed by Newmark's sliding-block approach (Newmark, 1965). This method can be also applied regionally. In these assessments, again, it is important to establish the material strengths, liquefaction susceptibility, and groundwater conditions.

25.6.5 Vertical ground displacements due to post-earthquake pore water pressure dissipation

Besides the lateral spread displacements occurring during and after an earthquake, saturated loose sands can settle (vertically deform) as the earthquake-induced excess pore pressures dissipate (typically called post-liquefaction settlements). Although not a major concern in pipelines located in generally flat areas, these settlements need careful attention in some situations. For example, plant structures, compressor stations, etc., located in soft/loose soil zones may require pile-supported foundations. Under such conditions, there is a potential for post-liquefaction settlements leading to major differential settlements between pile-supported structures and pipelines. Thus, provision should be made for adequate flexibility of pipeline to minimize potential pipeline breakage due to differential settlements.

Studies conducted to evaluate volume change after cyclic loading have shown that in general, the factors controlling post-cyclic settlements in sands are the degree of excess pore water pressure generation and the induced cyclic shear strain. Owing to the direct connection with excess pore water pressure development, potential for volumetric strains has been often linked with the field density. 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).

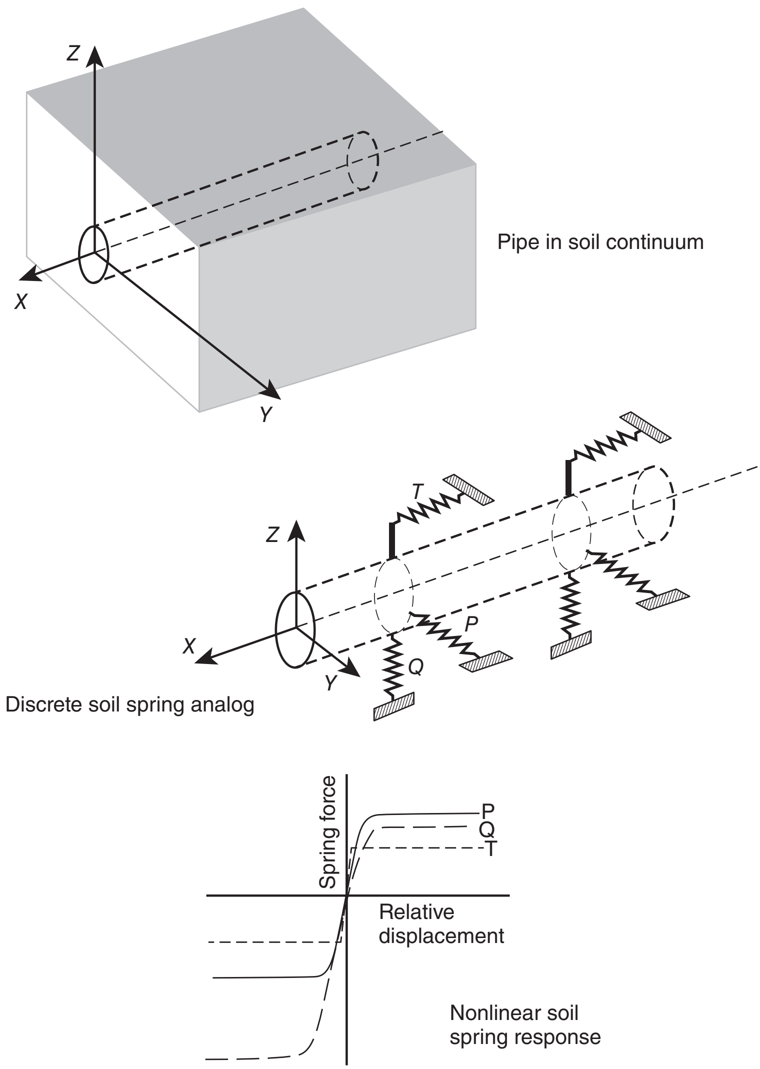
Much larger vertical movements are expected at river crossings, in the vicinity of dykes, ditches, road embankments, etc., due to distortion of the soil mass. Estimation of such vertical deformations would require rigorous site-specific analyses.

25.7 Pipeline response to earthquake hazards

The analytical assessment of buried pipeline response to permanent ground displacement dates back to the mid-1970s. Current recommendations on the methodology for assessing pipelines for ground displacement are contained in PRCI guidelines (Honegger and Nyman, 2004; PRCI, 2009). Nonlinear finite element techniques are the only practical means available to analyze all but the most simple of problems. Methods relating generic ground displacement values to rates of pipeline damage, such as those incorporated into HAZUS risk assessment software (FEMA, 2011) and presented in American Lifelines Alliance guidelines (ALA, 2001b), 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 mechanics of implementing a finite element analysis have remained largely the same over the last 30 to 40 years. The primary advancement in performing such analyses is the availability of powerful desktop personal computers and compatible nonlinear analysis software that accounts for material yielding and large deformations.

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 as illustrated schematically in Fig. 25.2. 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 nonlinear behaviour of soil



25.2 Representation of pipe-soil interaction using nonlinear spring elements.

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.

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 soil springs are defined in a global coordinate system. As a result of this, in 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. However, the error introduced by this misalignment is acceptable considering other assumptions and uncertainties inherent in the analysis.

In a case where loading and unloading of the soil springs occur, unloading characteristics of the soil springs need to be modeled. Relative pipe–soil displacements are permanent in a sense that the soil does not 'spring back'. Therefore, soil spring forces should quickly drop to zero or unload along a path parallel to the initial soil spring stiffness.

There is considerable uncertainty regarding the relationships used to compute soil spring properties. Most of this uncertainty is related to estimates of the soil strength parameters. The uncertainty in estimating soil strength parameters for pipeline analyses in fine-grained soils is further complicated by the fact that pipelines are typically located above the water table and within the desiccation zone of the soil. The strength of partially saturated desiccated soils is not well defined in soil mechanics.

Current analysis techniques assume the equivalent soil springs act independently, a common assumption for analytical representation of pile foundations and similar buried structures. This assumption can introduce errors related to the potential for different soil restraints for oblique soil displacement relative to the pipe (e.g., a combination of horizontal and vertical displacement) and the dependency on soil spring force at a particular point along the pipeline on adjacent relative pipe–soil displacement. The error associated with the assumption of independent soil springs is generally small relative to the overall uncertainty typically associated with defining the soil strength properties in the equivalent spring formulations.

For pipeline load conditions produced by an imposed displacement, relatively large strains can be accepted provided the pipeline is in good condition and the girth welds are capable of developing gross-section yielding of the pipe. For situations where the pipeline is subjected to high longitudinal compression strains from a combination of axial and bending loads, there is a potential for the development of local buckling of the pipe wall.

While continuum analyses hold the promise of eliminating many of the simplistic representations of soil–pipe interaction using pipe elements and soil springs, several significant obstacles remain to be overcome before continuum analysis methods can be considered superior to pipe element and soil spring representations for routine engineering applications:

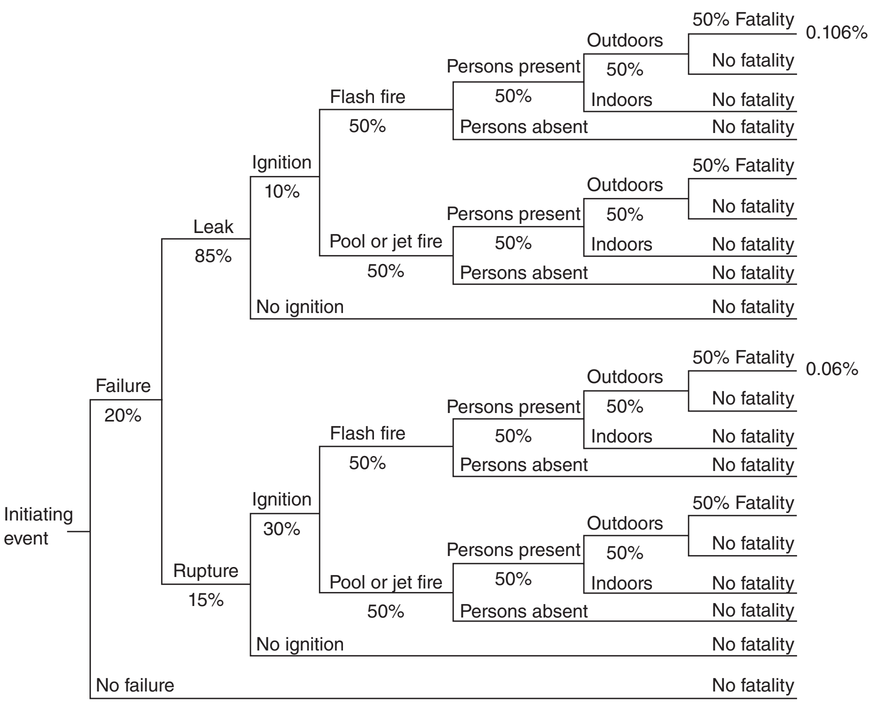
- It is typically necessary to represent several hundred meters of pipeline in the analytical model which results in an unwieldy large model that may take days to run using normally available computational resources.
- The complexity of the model makes it much more difficult to extract results of interest and incorporate relatively minor changes (e.g., change of soil cover or pipe diameter).
- The ability to model large relative displacements is often not possible without an efficient means to reformulate the model mesh.
- Continuum models are generally not capable of capturing flow and fracture behavior and the development of slip planes in the soil surrounding the pipeline.
- Modeling of the soil in a continuum models requires soil properties not normally available and often requires calibration of the analytical material model.
- The fidelity of results from continuum models is likely to be no greater than what is obtained from simple soil spring models, since available test data that could provide a basis for validation do not include information on the state of stress within the soil and at the pipe–soil interface.

25.8 Consequences of pipeline damage

The protocol developed for the California Department of Education (2007) provides a good overview of a relatively simple consequence assessment process based upon the ALOHA program that is generally available to the public (NOAA, 2011). Consequences of damage to an oil or gas pipeline are typically related to exposure to thermal radiation due to a pool fire, jet fire, or flash fire from a vapor cloud ignition. The consideration of explosion from natural gas release is highly unlikely unless unique conditions exist that would allow accumulation of gas near the ground without ignition, confinement of the gas cloud, and presence of an ignition source in the zone of confinement. Meteorological conditions (wind direction, wind speed, and atmospheric stability) play an important role in evaluating consequences as they directly impact the direction of gas dispersion, the air mixing that can dilute the natural gas to a point where ignition cannot happen, and the radiated heat distribution patterns from a subsequent fire. Seasonal conditions also affect the potential threat posed by ignition of brush.

Measures of consequences are generally established as a percentage of the lower flammable limit for vapor clouds or the likelihood of exposure to serious thermal radiation. These consequence measures can be linked with a specified likelihood for individual mortality. Flammable vapor clouds are often conservatively considered to pose a risk of ignition and mortality from flash fire if the concentration of flammable vapor exceeds 30–50% of the lower flammability limit. Onset of serious injury from thermal radiation is often considered to occur for exposures exceeding 16 000–18 000 kW/m^2 which corresponds to roughly a 1–15% chance of mortality.

A fault tree or event tree approach is commonly used to determine the likelihood of exceeding exposure limits for flash fire or thermal radiation. A simple representative event tree is illustrated in Figure 25.3 for a single



25.3 Example of an event tree assessing likelihood of a fatality from thermal exposure.

initiating event (e.g., slope failure) leading to a fatality. Using an event tree or fault tree approach requires assigning likelihoods to each branch of the tree along the event path of interest, as shown in Fig. 25.3.

25.8.1 Relating computed strain to loss of pressure integrity

For a seismic risk assessment, pipeline failure is related to the exceedance of a specified longitudinal strain limit, typically 2–4% for relatively modern (post-1960) welded steel oil and gas pipelines. One or more failure branches may be necessary to differentiate the consequences from seismic hazards that can produce different probabilities of leak versus rupture. For example, damage from large fault displacement or lateral spread displacement is much more likely to result in rupture than small displacements associated with liquefaction settlement.

25.8.2 Leak versus rupture

In general, opinions on pipeline failure of full bore pipeline rupture given exceedance of an ultimate strain limit range from 10% to 40%. For seismic risk assessment, the potential for full rupture should not be taken below 20% and be biased toward a higher likelihood based upon the degree to which the seismic ground displacements exceed the estimated pipeline displacement capacity.

25.8.3 Ignition likelihood

The likelihood for ignition is largely a function of the built environment near the pipeline. Ignition is more likely for pipelines located within the right-of-way of high-voltage transmission pipelines or urban areas with above-ground electrical lines. Opinions of ignition range from 10% to over 30%, depending upon these factors and the amount of flammable that can potentially be released.

25.8.4 Fire hazard from leak

The exposure to a flash fire from a pipeline leak is dependent upon the size of the leak which is generally assumed to be equivalent to a hole with a diameter of 25–50 mm. In addition to the rate of material release from a leak, the orientation of the leak can significantly alter the exposure for high-pressure pipelines. Caution is warranted when using simple programs such as ALOHA which assume vertical release from pipelines. Proprietary software capable of more refined modeling of leak conditions may be necessary to capture the effects of impeded flow or influence of jet direction on thermal exposure.

25.9 Mitigation approaches to reduce risk to pipelines

Upon identification of the geotechnical hazards and the resulting vulnerability of a given structure, a combination of structural retrofitting and/or geotechnical remediation (ground improvement) is often considered in the design of mitigation measures. 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. 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 seismic hazard. Other than rerouting the pipeline to avoid the hazard, there are two methods to reduce pipeline exposure. 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. In rare instances, aerial crossings can be used to avoid hazards of limited size. Isolation of pipeline from geotechnical hazard is also considered favorable in certain situations. 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 to 'slide past' or 'slide below' 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 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 extreme difficulties in obtaining new right-of-way for existing pipelines.

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).

When ground improvement is considered to be the desired option, 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. Historically, ground improvement has been used as a means of improving the post-construction bearing capacity and settlement performance of soils under static loading conditions, and a variety of ground improvement techniques have evolved in the past few decades. In addition to resisting static loads, some of the ground improvement measures have been effectively used to retrofit facilities that are located within, or that have foundations supported on liquefiable soils. 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 Fig. 25.4). 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 a 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 10–12 m below existing ground level. Below this depth, the achieved improvement in penetration resistance diminishes considerably.

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. The ground improvement configurations used in practice are

25.4 Ground densification using vibro-replacement at a river bank in Vancouver, British Columbia, Canada, to reduce the risk of liquefaction-induced lateral ground displacements (Wijewickreme *et al.*, 2005).

clearly dependent on geotechnical risks that are to be mitigated, and these configurations essentially fall into one of the following two categories: (i) in-ground densified barrier(s) aligned perpendicular to the direction of ground movement to reduce liquefaction induced lateral spreading; and (ii) densification of wide-area footprints beneath and/or around foundation footprints to improve bearing capacity and to reduce the impacts from lateral spreading.

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 are required in selecting the parameters and testing tools to assess the conformance of ground improvement to specified criteria. In addition, effects from parameters that affect the soil behavior (e.g., aging 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 is essential. Furthermore, structural evaluation of the performance of structures based on data from monitoring as well as modification of ground improvement methodology and configurations to meet the constraints are generally required.

Prediction of the anticipated geotechnical hazard and liquefaction-induced permanent ground displacements are critical considerations in the design of remedial measures. These predictions are often undertaken using approaches combining both numerical and empirical methods.

25.10 Challenges and issues

25.10.1 Pipeline strain versus size of rupture

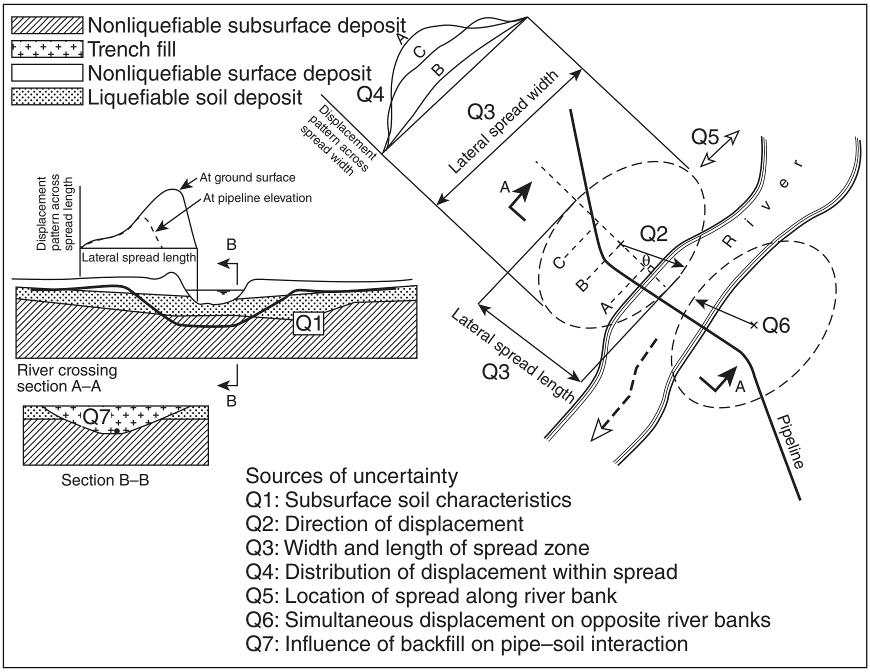
Establishing pipeline strain criteria that relate pipeline strain to a probability for loss of pressure integrity is an ongoing area of active research (PRCI, 2011). For new pipelines, establishing acceptable tension strain limits is often based upon the results of full-scale or wide-plate testing of girth welds with or without machined flaws to represent defects that may be missed by weld inspection procedures. However, the number of tests performed is generally not sufficient to provide a statistical relationship between tension strain and failure probability. There are no generally accepted methods for relating pipeline strain to the severity of the pipeline failure mode. As a result, establishing strain criteria must be based upon expert judgment.

25.10.2 Geotechnical constraints

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 behavior of soil (e.g., stress and strain level dependence, effects of particle fabric, effects of loading paths.) present additional constraints to the level of accuracy attainable in seismic performance evaluations. With respect to lateral spread displacement hazards, some of the basic uncertainties in defining the hazard are illustrated in Fig. 25.5. Sensitivity of the geotechnical hazard estimates to the above considerations along with the uncertainties arising due to the difficulties in accurately defining earthquake shaking (i.e., ground acceleration time histories) should be kept in mind when interpreting outcomes from hazard assessments.

25.10.3 Condition of older pipelines

A key difficulty in assessing the vulnerability of existing pipelines is the uncertainty in estimating the tension strain capacity of the existing girth welds. Compression strain capacity is governed by buckling of the pipe wall and is not highly dependent upon girth weld quality. For older pipelines,



25.5 Uncertainties associated with quantifying lateral spread displacement hazards.

there will not be sufficient information on the severity or prevalence of girth weld defects that could reduce pipeline tension strain capacity. This difficulty is normally addressed by simply assuming a tension strain capacity based upon assumed flaw severity. Another alternative, if portions of the pipeline are removed to provide some pipe weld information, is to assume the removed pipe is identical to the remaining pipe and assign a strain capacity based upon a fitness for purpose assessment. API 1101 Appendix A (API, 2010) and API 579 (API, 2007) provide methodologies for defining girth weld strain capacity, but these methods can be extremely conservative and require special knowledge and experience to implement correctly.

25.10.4 Acceptable risk

As risk is defined as the product of vulnerability and consequences, a 0.01% (1 in 10000) chance per year of an event causing 1000 deaths is equivalent to a 10% chance per year of an event causing the death of one person. Therefore, risk alone is generally insufficient for making decisions where the general public is involved because it is extremely difficult to educate the public to accept serious consequences, such as death. Conversely, the public is more accepting of severe consequences for events that are deemed sufficiently rare as to be considered ‘acts of God’. Thus, there is more outrage expressed if a pipeline fails under normal operating pressure than if it fails as a result of a landslide or an extreme flood event, even if the annual likelihood of failure for the three causes is the same.

The difficulty in conveying risk-based decisions to the public at large is often ameliorated by an official governmental regulatory process in which specific risk-based acceptance criteria are developed (e.g., ASCE, 2010), often with the advice and input from experts. That is not to imply that the regulatory requirements are always based on a rational interpretation or assessment of risk. As with any governmental regulatory action, the process can be subverted by individuals and organizations acting in their own best interests. The benefit of governmental regulations establishing risk acceptance criteria is primarily that the regulations can impose uniformity on those performing the risk assessments.

25.10.5 Risk-based code provisions

At this time, there are no code requirements on the level of reliability that is required for the design of oil and gas pipelines. Work sponsored by the Pipeline Research Council International (PRCI) has led to a set of guidelines for implementing a reliability-based design framework with a recommendation with respect to performance goals (Nessim and Zhou, 2005). This work has been largely incorporated into the non-mandatory Annex O of CSA Z662. Those referring to Annex O should be aware that the reliability limits contained therein are only recommendations and not requirements. Considerable caution should be taken in considering adoption of the reliability limits in Annex O for several reasons:

- The Annex O reliability targets were developed using the reliability under normal operating conditions of the population of existing pipelines as a baseline value of acceptable risk. A more appropriate approach would have been to base non-seismic reliability targets on the minimum performance allowed in pipeline codes (ASME, 2007).
- The Annex O reliability targets are based upon pipeline diameter, pressure, and population density and typically require an annual likelihood of pipeline failure in the range of 1/150 000 to 1/200 000 which is unreasonable compared with other reliability targets where the potential for death and injury is far greater than what exists for oil and gas pipelines.
- Development of the reliability targets did not address the difference in performance expectations for extreme rare events. This is especially relevant to seismic risk assessment. For example, design requirements

for typical buildings in the 2010 version of standard ASCE 7 are based upon a target annual probability of sudden progressive failure of 7/10 000 000 for non-seismic loading and 1/24 750 for seismic loading (ASCE, 2010).

As an alternative to the PRCI reliability targets, consideration could be given to adopting an acceptable rate of pipeline failure from seismic loading be no greater than 1/24 750, the same as the targeted annual probability of building collapse from earthquake loading adopted in current US building standards (ASCE, 2010).

25.11 Future trends

25.11.1 Numerical modeling

The use of numerical modeling, in general, allows the effects of a wide range of variables to be assessed in a timely and efficient manner. 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).

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). Work in this area is expected to continue.

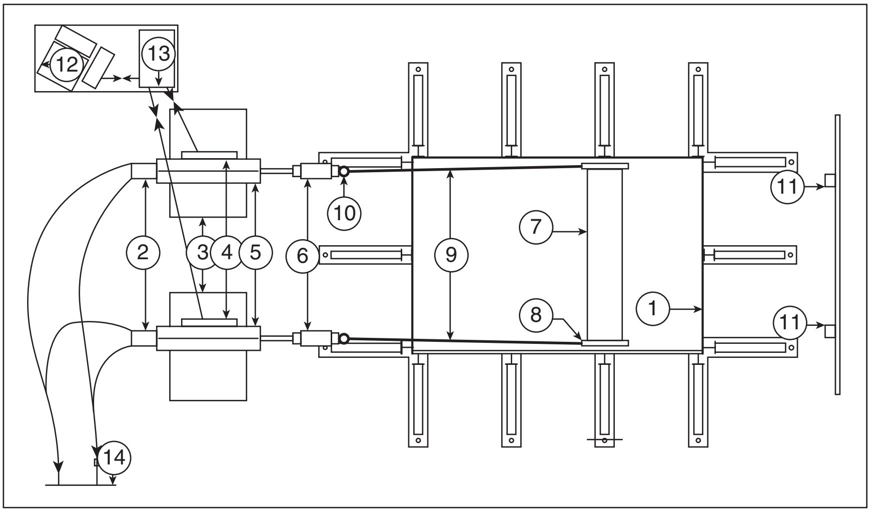
25.11.2 Analytical approaches to capture nonlinear pipe material behavior of plastic pipelines

Since its introduction in the late 1960s, polyethylene (PE) pipes (either high-density polyethylene (HDPE) or medium density polyethylene (MDPE)) have be come increasingly popular in natural gas distribution networks due to their lower material, installation and maintenance costs, corrosion resistance, lower friction at the interface, lightweight, and claimed greater capacity to accommodate displacements than its counterpart, 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).

Over the years, numerous studies have been performed on steel pipes to characterize the behavior when the pipes are subjected to ground movement. However, reported experimental 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 in comparison to steel, there is likelihood for significant limitations to arise when methods developed for steel pipes are used in evaluating the response of PE pipes. Clearly, 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. With this background, 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).

25.11.3 Full-scale model testing

Modeling of chosen full-scale pipeline configurations in the laboratory provides a very attractive way of capturing and understanding the complexities associated with soil–pipe interaction. Due to the large number of variables, full-scale testing provides a meaningful approach to characterize soil-springs for pipe–soil interaction modeling (i.e., provides a meaningful approach to estimate parameters of soil-springs in axial, lateral, and upward directions to model the interaction between pipe and soil). Physical models simulating the field situations also play a key role in calibrating and validating the analytical approaches and numerical models. (Note: tests in smaller scales, however, may be subjected to errors associated with scaling.) 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 Figs 25.6 and 25.7 for details related to the University of British Columbia (UBC) test facility).



Test system components

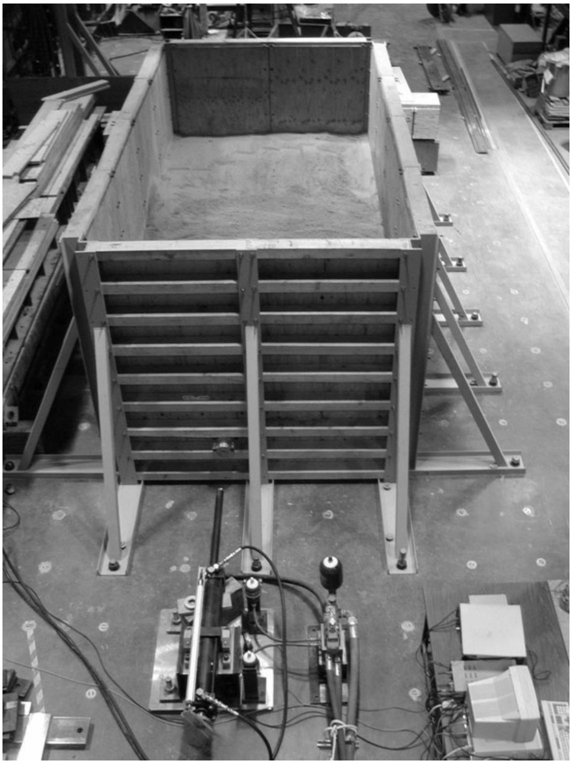
No.	Description	No.	Description
1	Soil test chamber (3.78 m x 2.5 m)	8	3 bolts end clamps
2	Servo controller	9	1 1/8" steel cables
3	Pedestal	10	Shackles
4	Linear voltage displacement transducer	11	String potentiometers
5	Hydraulic actuator	12	Data acquisition system & computer
6	Load cell	13	Control system
7	Steel pipe (2.4 m long)	14	To hydraulic power

25.6 Layout of Physical Modeling Facility at UBC (set-up for lateral soil-pipe restraint testing) and test components.

25.11.4 Field monitoring and testing of pipeline performance

Field monitoring of the performance of pipelines, or soil mass in the vicinity of pipelines, has the potential to play an important role with respect to several key aspects of pipeline geotechnical engineering including, but not limited to the following:

- assessment of potential imminent pipeline concerns;
- identification of pipeline performance in areas of less understood, varying, soil and groundwater conditions;
- development of an understanding of field variations that are not necessarily captured in general design works using existing guidelines;



25.7 View of Physical Modeling Facility at UBC (set-up for axial soil–pipe restraint testing.

- development of real-life data for validation of outcomes from numerical modeling of pipe–soil interaction problems;
- furthering the understanding of observations from physical modeling.

Under certain circumstances, continuous field monitoring of critical pipeline sections becomes valuable. This may become essential in situations where numerical modeling is complicated due to lack of knowledge in representing the slide fronts and due to difficulties in determining the actual pipe–soil interaction taking placing because of varying soil conditions. Soil constitutive laws, boundary conditions, and the assumptions in initial soil stress state can significantly affect the results of numerical modeling. However, in remote areas or in zones where the ground movement is spread over a wide region, the continuous monitoring of the pipeline would become costly and unfeasible. An advantage in field monitoring is that they could be conducted over longer periods, without being subjected to space–time constraints that are more common in laboratory environments.

25.12 Conclusions

The preceding discussion provides a general overview of key aspects of quantitative seismic risk assessment for oil or gas pipelines. As is hopefully apparent from this discussion, the process requires numerous assumptions regarding many poorly defined or highly variable input parameters and models. Therefore, specific results can be highly dependent upon the professional qualifications and experience of the personnel making the assumptions and conducting the analyses. This level of uncertainty is considered to be one of the main reasons why explicit risk acceptance criteria have not been directly incorporated regulatory requirements. Those undertaking or contemplating facilitating decisions on oil and gas pipelines based upon the results from a seismic risk assessment should exercise caution in giving undue credence to numerical risk values. Given the current state of knowledge, estimates of risk within at least a factor of two can be considered 'equal'.

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