Analysis and Design of Buried Steel Water Pipelines in Seismic Areas[1]

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Abstract: This paper offers an overview of available methodologies and provisions for the structural analysis and mechanical design of buried welded steel water pipelines subjected to earthquake action. Both transient (wave shaking) and permanent ground actions (from tectonic faults, soil subsidence, landslides, and liquefaction induced lateral spreading) are considered. In the first part of the paper, following a brief presentation of seismic hazards, modeling of the interacting pipeline soil system is discussed in terms of either simple analytical models or more rigorous finite elements, pinpointing their main features. The second part of the paper outlines pipeline resistance, with emphasis on the corresponding limit states. Possible mitigation measures for reducing seismic effects are presented, and the possibility of employing gasketed joints in seismic areas is discussed. Finally, the discussed analysis methodologies and design provisions are applied in a design example of a buried steel water pipeline located in an area with severe seismic action. **DOI: 10.1061/(ASCE)PS.1949-1204.0000280.**

Introduction

The structural performance of steel water pipelines in geohazard areas is an issue of increasing interest. In the particular case of seis mic action, the main purpose of pipeline operators is to minimize seismic risk to the pipeline, safeguarding the unhindered flow of water resources following a severe seismic event. To this purpose, the structural damage of the steel pipe should be minimized in order to maintain the structural integrity of the pipeline and prevent loss of water containment.

Earthquake actions in buried steel pipelines can be classified into two main categories: (1) transient actions, associated with wave shaking phenomena; and (2) permanent ground induced deformations, such as seismic faults, landslides, subsidence settle ments, and liquefaction induced lateral spreading. Past earthquakes have induced significant damage in buried pipelines, attributed to both transient and permanent ground deformations (EERI 1999; Liang and Sun 2000; O'Rourke 2003). These reports indicated that damage due to permanent ground induced deformations typically occurs in specific areas with severe ground motion and is associated with high damage rates, whereas damages due to seismic wave ac tion occur over substantially larger areas but are associated with lower damage rates.

The vast majority of research publications referring to the seis mic analysis and design of buried steel pipelines has been driven by the need of safeguarding the integrity of hydrocarbon (oil and gas) pipelines. Kouretzis et al. (2006) presented a more detailed literature review of transient ground induced actions, whereas Vazouras et al. (2010, 2012) provided a complete summary of pre vious works on permanent ground induced actions on buried pipe lines. Extensive experimental, analytical, and numerical research on the effects of permanent ground induced actions on the struc tural integrity of buried steel pipelines has been conducted in the course of the Safety of Buried Steel Pipelines Under Ground Induced Deformations (GIPIPE) project (Karamanos et al. 2015a; Vazouras et al. 2015; Sarvanis et al. 2016). This research project has performed large scale experiments, supported by extensive numerical simulations, and developed simple and efficient analyti cal methodologies. It is worth noticing that current water pipeline design standards or manuals, such as the American Water Works Association (AWWA) manual M11 (AWWA 2004), do not contain provisions for seismic design.

Several important differences exist between hydrocarbon and water pipelines, so that direct application of design guidelines and tools developed for oil and gas pipelines to water pipelines may not be appropriate. Steel water pipelines are different from hydrocar bon steel pipelines in several ways:

- They are considerably thinner, with much higher values of D:t ratio;
- They are made of lower steel grade; X42 or X46 are usual grades for water steel pipes, whereas onshore hydrocarbon pi pelines use X70 steel grade or higher;
- They have different type of joints; oil and gas pipelines almost exclusively use butt welded full penetration joints, whereas water pipelines are constructed with welded lap or gasketed joints;
- They operate under lower pressure, which does not exceed 50% of yield pressure; this may not be necessarily beneficial, given the fact that in most cases the presence of internal pressure may prevent cross sectional distortion, increasing pipeline deforma tion capacity; and

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- They contain special components (e.g., elbows and junctions) with a different geometry and configuration than the corre sponding components in oil and gas pipelines.

The main seismic design requirement is that pipeline seismic actions should be less than the corresponding pipeline resistance. This paper offers an overview of seismic analysis and design of buried welded steel pipelines for water transmission and distribu tion, based on existing information in the literature and in relevant codes, standards, and design guidelines. Following an outline of existing provisions in pipeline design standards and recommenda tions in North America and Europe, the paper refers to seismic ac tions, due to both transient and permanent ground deformations. The second part of the paper presents issues related to pipeline re sistance, with direct reference to possible failure modes. Possible measures for mitigating seismic effects on buried pipelines are also discussed. Finally, a design example that illustrates the application of the above methodologies and design provisions is presented.

Existing Standards and Recommendations for Pipeline Seismic Design

The ASCE (1984) guidelines were the first document that trans ferred and adjusted existing knowledge and design tools of seismic engineering into the earthquake analysis and design of buried pipe lines. In particular, the document was based mainly on relevant work by N. M. Newmark, W. J. Hall, and their associates at the University of Illinois (e.g., Newmark 1967; Newmark and Hall 1975). This document was the basis for the American Lifelines Alliance (ALA) guidelines (ALA 2005), which contain the most complete set of provisions for this subject. Some of the ALA (2005) provisions are used in this paper. The ALA guidelines also constituted the basis for the Indian National Information Centre of Earthquake Engineering (NICEE) guidelines (NICEE 2007) for earthquake design of buried pipelines.

The Pipeline Research Council International (PRCI) guidelines (PRCI 2004) for pipeline earthquake design and assessment can be considered as an update of the ASCE (1984) guidelines for buried pipelines transporting natural gas and liquid hydrocarbons. In par ticular, they accounted for more recent research on soil loading on buried pipelines and on strain based pipeline limit states, and pro posed more advanced tools for pipeline stress analysis. More re cently, PRCI published design guidelines for the design of oil and gas guidelines in landslide areas (PRCI 2009), which adopt analysis and design methodologies similar to those proposed in PRCI (2004).

American Society of Mechanical Engineers (ASME) B31.4 (ASME 2006) and ASME B31.8 (ASME 2006) standards, widely used for oil and gas pipeline design, respectively, state that earthquake loading should be considered in pipeline design as an accidental (environmental) load. Nevertheless, those standards do not contain information on how seismic action on the pipeline should be com puted. Similarly, Canadian Standard Association standard Z662 (CSA 2007) specifies fault movements, slope movements, and seismic related earth movements as additional loading that should be taken into account for pipeline design, but does not provide any further information on how those actions should be quantified.

European standard Comité Européen de Normalisation (EN) 1594 (CEN 2000) has been a popular standard for the general design of high pressure gas pipelines. Annexes D and E of this stan dard refer to landslide and high seismicity areas, respectively; both Annexes suggest that these geohazards should be taken into account in pipeline analysis and design, and some mitigation measures are proposed. Similarly, European standard EN 16416, also known as International Organization for Standardization (ISO) 13623 standard (CEN 2003), in subsection 6.3.3.3 provides general information and suggestions on seismic design. European standard EN 1998 4 (CEN 2006) provides guidance for the earthquake analysis and design of buried pipelines. This standard was developed primarily for the seis mic design of liquid storage tanks; limited information on buried pipelines is contained in Chapter 6 and Annex B. Furthermore, EN 1998 4 is intended to cover all possible materials (steel, concrete, plastic), and therefore it may not be a standard suitable for the seis mic design of buried steel pipelines. However, some clauses of EN 1998 4 can be useful for pipeline design and are employed in this paper. Finally, among numerous national standards for pipeline design, the Dutch Nederlands Normalisatie Instituut (NEN) standard 3650 (NEN 2006) is highlighted; despite the fact that earthquake action may not be an issue in The Netherlands, NEN 3650 contains important information for ground induced actions on pipelines, es pecially for soil pipe interaction in settlement areas.

Seismic Actions in Continuous Buried Pipelines

Ground induced earthquake actions on buried pipelines can be cat egorized as (1) transient actions and (2) permanent deformations. Transient actions are caused by wave shaking effects, whereas per manent ground deformations are due to fault movements, landslide activation, and liquefaction induced lateral spreading. This section examines the effects of ground induced earthquake actions on con tinuous steel buried pipelines. Those are welded pipelines with welded lap joints; butt welded connections are employed only in a few instances.

Transient Action

Transient action is often referred to as *wave propagation hazard* and is characterized by peak ground acceleration and velocity, as well as by the appropriate propagation velocity. It is caused by ground shaking due to body and surface seismic waves travel ling within the soil. Body waves (compressional and shear) propa gating through the three dimensional ground are generated by seismic faulting at the seismic source. Surface waves (Love and Rayleigh) travel along the ground surface and are generated by the boundary condition imposed by ground surface to body waves.

Seismic wave action analysis of a buried pipeline is a complex problem requiring wave propagation analysis on the three dimensional soil pipe system, accounting for the soil pipe inter face. As an alternative, the simplified method developed by Newmark (1967) can be employed, which estimates soil strain and curvature due to a traveling wave of constant shape in terms of peak ground motion parameters. This method expresses the maximum ground strain ε_g in the direction of wave propagation by the following equation:

$$\varepsilon_g = \frac{\mathrm{PGV}}{C} \tag{1}$$

where PGV = peak ground velocity, which is the maximum hori zontal ground velocity in the direction of wave propagation; and C = apparent velocity of the seismic wave. The maximum axial force on the pipeline can be computed as the minimum value of F_1 and F_2, defined as follows (ALA 2005):

$$F_1 = EA\varepsilon_g \tag{2}$$

and

$$F_2 = (t_u \lambda)/4 \tag{3}$$

where t_u = ultimate frictional force of soil per unit pipe length, act ing on the pipe in the axial direction; and λ = corresponding wavelength at pipe location. In addition, the maximum ground curvature, k_g, can be computed as the second derivative of the transverse displacement with respect to the axial coordinate along the pipe, resulting in the following equation:

$$k_g = \frac{\mathrm{PGA}}{C^2} \tag{4}$$

where PGA = peak ground acceleration, which is the maximum ground acceleration perpendicular to the direction of wave propagation. The peak ground motion parameters PGV and PGA can be obtained from seismic records available in the area of interest from relevant seismic maps or following a dedicated local geotechnical analysis. Furthermore, several arguments exist concerning the choice of the value of C in Eqs. (1) and (4). For a site subject to body wave propagation, the value of C should be taken in the range of 2,000 to 5,000 m/s according to O'Rourke and El Hmadi (1988), whereas the ALA (2005) guidelines suggest a value equal to 4,000 m/s (13,000 ft/s), which is within the range suggested by O'Rourke and El Hmadi. However, in certain cases where Rayleigh waves are important, it may be necessary to consider lower propagation velocities, typically as low as 500 m/s, which is a lower (conservative) bound for the apparent (effective) wave velocity, as reported in the analysis by Trifunac and Lee (1996). This is in agreement with the peak ground strain versus PGV data reported by Iwamoto et al. (1988) and Paolucci and Smerzini (2008), also summarized by O'Rourke and Liu (2012). These data show that the lowest inferred C values lie between 500 and 1,000 m/s. These low C values are representative for low velocity soft sedimentary formations (e.g., Holocene sediments), whereas higher values should be employed for sites located on stiffer formations (e.g., older sediments or bedrocks).

Permanent Ground Actions—Analytical Methods

A significant amount of earthquake damage to steel pipelines has been attributed to permanent ground deformations (fault movements, landslides, soil subsidence, and liquefaction induced lateral spreading). Permanent ground deformations are applied on the pipeline in a quasi static manner, and they are not necessarily associated with severe seismic events; however, under those actions the pipeline may be seriously damaged.

Fault Movement

An active tectonic fault constitutes a discontinuity between two portions of the bedrock, along which relative motion of the two portions may occur. An active tectonic fault is a planar fracture or discontinuity in a volume of rock, across which significant displacement may occur as a result of earth movement. The movement is concentrated in a rather narrow fault zone and can be horizontal (strike slip fault) or vertical (normal or reverse fault), as shown in Fig. 1, and also can be in an oblique direction (oblique fault). It is possible to estimate peak ground displacement of a fault, PGD_F, in terms of earthquake moment magnitude using empirical relations, such as the equations proposed by Wells and Coppersmith (1994).

Subsequently, the axial strain induced in the pipeline wall can be estimated using the analytical procedure developed by Kennedy et al. (1977). For the case of horizontal (strike slip) faults [Fig. 2(a)], which employs the horizontal ground induced displacement PGD_{FH}. According to this methodology, the axial strain caused by pipeline stretching ε_m, referred to as *membrane strain*, can be computed as follows:

$$\varepsilon_m = \frac{\mathrm{PGD}_{FH}}{L_H}\cos\theta + \frac{2}{3}\left(\frac{\mathrm{PGD}_{FH}}{L_H}\sin\theta\right)^2 \tag{5}$$

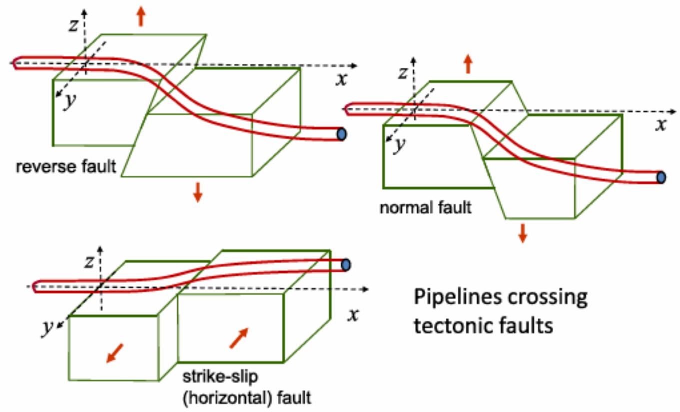

Fig. 1. Schematic representation of pipeline configuration crossing tectonic faults

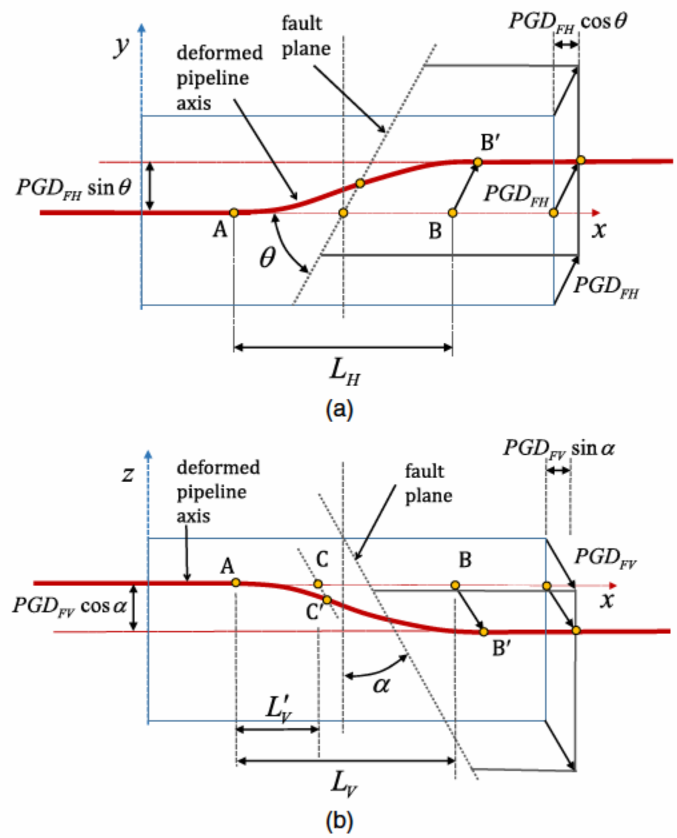

Fig. 2. Pipeline deformation crossing: (a) horizontal fault at angle θ (plan view); (b) normal fault at angle α (side view)

where θ = angle between the fault plane and the pipeline axis; and L_H = distance between the two ends of the S shaped pipeline configuration. The first term is linear and is due to the fault motion component in the direction of the pipeline axis. The second term is quadratic due to axial stretching because of pipeline transverse deformation.

A similar equation exists in the ALA (2005) guidelines

$$\varepsilon_m = 2\frac{\mathrm{PGD}_{FH}}{L_H}\cos\theta + \left(\frac{\mathrm{PGD}_{FH}}{L_H}\sin\theta\right)^2 \tag{6}$$

Comparison of Eqs. (5) and (6) indicates that the latter contains an additional factor of 2, which is aimed at accounting for the un certainties of the methodology of Kennedy et al. (1997). It is im portant to notice that axial deformation of the pipeline extends well beyond the S shaped pipe segment and that Eqs. (5) and (6) refer only to axial deformation (stretching) of the S shape of the pipe.

For an oblique fault with fault movement PGD_{FV} in the vertical direction and PGD_{FH} in the horizontal direction, the axial strain in the pipeline is

$$\varepsilon_m = \frac{PGD_{FH}}{L_H}\cos\theta + \frac{2}{3}\left(\frac{PGD_{FH}}{L_H}\sin\theta\right)^2 + \frac{2}{3}\left(\frac{PGD_{FV}}{L_V}\right)^2 \quad (7)$$

where L_V = distance between the two ends of the S shaped pipeline configuration, shown in Fig. 2(b). A deficiency of the above ana lytical methodologies is that they do not provide a reliable meth odology for determining the values of L_H and L_V.

Sarvanis and Karamanos (2016) presented a more elaborate, yet very efficient, analytical methodology for determining the strain in buried pipelines at fault crossings. This methodology employs an assumed shape function, is applicable to both horizontal and nor mal faults, and provides a systematic procedure for the calculation of lengths L_H and L_V in terms of soil conditions.

Furthermore, it is important to note that Eqs. (5) and (7) refer only to pipeline stretching, and neglect pipeline bending re sistance, which can be important. For the case of a normal fault, with fault movement PGD_{FV} in the vertical direction, Sarvanis and Karamanos (2016) proposed the following analytical expres sion for the maximum bending strain:

$$\varepsilon_b = \left(\frac{\pi^2}{8}\right)\frac{D}{L_V' L_V}(PGD_{FV}) \quad (8)$$

where L_V' = distance from the end of the S shaped configuration to the inflection point [Fig. 2(b)].

Finally, these analytical equations should be used in cases where the pipeline alignment in the fault area is straight, without bends. Bends are significantly more flexible than are straight pipes and exhibit significant stress and strain concentrations. The presence of bends near the fault zone may significantly affect pipeline stress and strain; in such a case, the previous analytical expressions for strain may not provide reliable predictions, and the use of a numeri cal finite element model is recommended for pipeline analysis.

Landslides

Landslides are associated with massive ground movements caused by soil slope instability [Fig. 3(a)]. The primary driving force for a landslide is soil gravity, but a seismic event may trigger this phenomenon. Numerous empirical methodologies have been re ported to determine the occurrence a landslide in terms of the dis tance from the epicentre and the magnitude of the earthquake event. To quantify the effects of landslide on pipeline deformation, the expected landslide movement PGD_S is required, and this can be estimated by available analytical expressions (Jibson 1994).

In the case of permanent ground induced action in the longitu dinal direction due to landslide, the pipeline should be designed for an axial force F, which is the minimum of F_1 and F_2, expressed in the following equations proposed by ALA (2005) guidelines:

$$F_1 = \sqrt{EAt_u(PGD_S)} \quad (9)$$

and

$$F_2 = (t_u L_S)/2 \quad (10)$$

where t_u = maximum (ultimate) frictional force of soil per unit pipe length acting on the pipe in axial direction; and L_S = length of pipe in soil mass undergoing movement. According to ALA (2005), the value of L_S may range between 100 and 250 m.

In the case of permanent landslide action in the transverse direction, the bending strain in the pipeline can be estimated by the following expression, assuming a cosine function of the pipe deformation:

$$\varepsilon_b = \frac{\pi^2 D(PGD_S)}{W^2} \quad (11)$$

where W = width of the landslide zone, ranging between 150 and 300 m, according to ALA (2005). Alternatively, assuming a beam with both ends fixed and a uniform lateral load p_u, the bending strain is

$$\varepsilon_b = \frac{p_u W^2}{3\pi E t D^2} \quad (12)$$

Furthermore, transverse permanent ground actions also induce axial tensile strains due to pipeline stretching.

Lateral Spreading

Lateral spreading is a consequence of liquefaction in a sandy soil layer; the soil loses its shear strength, resulting in lateral move ment of the liquefied soil, primarily in the horizontal direction [Fig. 3(b)]. In liquefaction induced lateral spreading, if the pipeline is contained in the liquefied layer, buoyancy should be taken into account, along with the horizontal ground movement imposed to the pipeline. To estimate permanent ground displacement due to liquefaction, PGD_L, several methodologies have been proposed

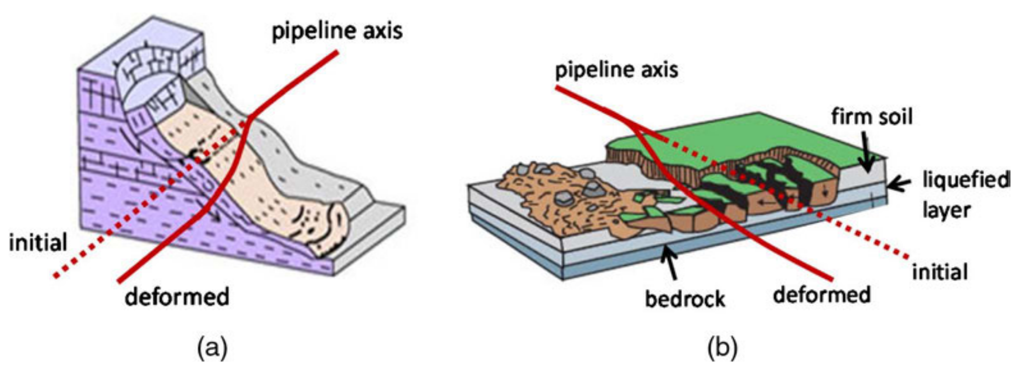

Fig. 3. Schematic representation of (a) pipeline configuration in the boundary of landslide; (b) liquefaction induced lateral spreading [(a and b) adapted from USGS 2004]

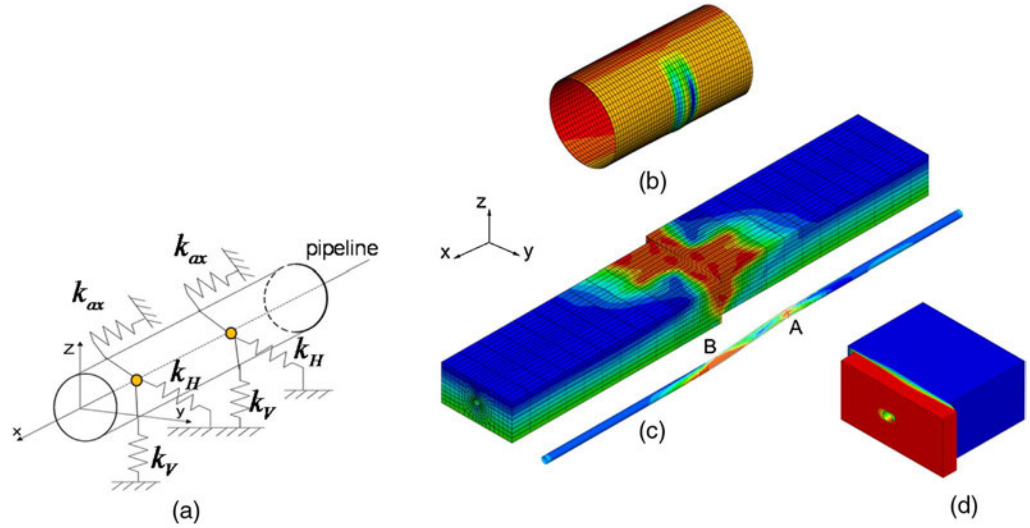

Fig. 4. (a) Level 1 of pipeline modelling: pipe (beam type) finite elements and soil springs attached to pipeline nodes in the three principal directions; (b d) Level 2 of pipeline modelling: shell elements and solid elements (adapted from Vazouras et al. 2010)

(e.g., Bardet et al. 2002). For longitudinal action, the corresponding maximum axial force in the pipeline can be calculated through Eqs. (9) and (10), whereas for transverse lateral spreading action, the maximum bending strain can be computed from Eqs. (11) and (12), replacing PGD_S with PGD_L.

Permanent Ground Deformation—Finite Element Modeling

Finite element modeling is a more rigorous tool for simulating the effects of ground induced actions on a buried pipeline. The finite element analysis of buried pipelines requires some computa tional effort and expertise, but offers an advanced tool for determin ing stresses and strains within the pipeline wall with significant accuracy with respect to the analytical formulas described previ ously. Two levels of finite element modeling exist, and are briefly described in the following subsections. Level 1 is adequate for regular design purposes, whereas Level 2 is used only in special cases where increased accuracy is necessary.

Level 1: Beam-Type Finite Element Analysis

This type of finite element analysis models the pipe with beam type one dimensional finite elements. These models have been used mainly for simulating permanent ground induced actions on pipe lines, but can be used for modeling wave effects as well. The finite element mesh near discontinuities (e.g., fault plane) should be fine enough to accurately describe gradients of stress and strains [Fig. 4(a)].

Type of finite elements: The use of regular beam elements for the pipeline model is not recommended because they cannot ac count for pressure. Instead, *pipe elements* are preferable for pipeline seismic analysis. These are enhanced beam type elements that ac count for the effect of hoop stress due to pressure. However, pipe elements usually have a circular cross section and do not describe cross sectional ovalization. Therefore the use of more elaborate pipe elements capable of describing cross sectional ovalization, sometimes referred to as *elbow elements*, can further improve the accuracy of the finite element model, especially at pipe bends (Bathe and Almeida 1982; Karamanos and Tassoulas 1996). Alternatively, it is possible to employ regular pipe elements, which are essentially beam elements with circular cross sections, accounting for ovalization effects at pipe bends through the use of appropriate flexibility factors and stress intensity factors.

Pipe and soil modeling: Pipe material should be modelled as elastic plastic, considering strain hardening. The ground surround ing the pipeline should be modelled by nonlinear springs [Fig. 4(a)] attached to the pipe nodes and directed in the transverse directions (with stiffness k_V and k_H in the vertical and lateral directions, respectively) and axially (k_{ax}). The springs should account for possible slip between the pipe and the soil. Expressions for these soil stiffnesses are offered in ALA (2005), based on the type of soil. Alternative expressions for those springs also can be found in the NEN 3650 standard Xie et al. (2013) and Saiyar et al. (2016) present further analysis of the limitations of soil spring reaction models, especially for the case of flexible pipes. In addition, com parison of pipe and elbow element methodologies with more rig orous finite element methodologies and experimental data have been reported recently by Sarvanis et al. (2016) and Sarvanis and Karamanos (2016).

Analysis procedure and output: To perform pipeline analysis under permanent ground induced actions, the imposed soil dis placements should be applied at the ends of the soil springs. The analysis follows three steps: (1) gravity, (2) operational loading (pressure and temperature), and (3) PGD application. The analysis output consists of stress resultants in pipeline cross sections and of the stresses and strains in the longitudinal direction. If the finite elements are not capable of accurately describing cross sectional distortion, the stresses and strains obtained may be quite different from the real stresses and strains in the pipeline wall, especially when the pipe wall begins to wrinkle due to local buckling. Con sideration of local stresses due to pipe wall wrinkling locations re quires a more detailed analysis, with the use of shell elements for modeling the pipe.

Level 2: Three-Dimensional Finite Element Analysis

Three dimensional finite element models constitute a rigorous numerical tool to simulate buried pipeline behavior under PGD. Such a model can describe in a rigorous manner the nonlinear geometry of the deforming soil pipe system (including distortions of the pipeline cross section), the inelastic material behavior for both the pipe and the soil, and the interaction between the pipe and the soil. However, it requires computational expertise.

Discretization: An elongated prismatic model is considered, where the steel pipeline is embedded in the soil, as shown in Fig. 4(b) for the case of a strike slip fault. Shell elements are employed for modeling the steel pipeline segment, and three dimensional *brick* elements are used to simulate the surrounding soil. The discontinuity plane (e.g., fault plane, edge of landslide or lateral spreading) divides the soil block in two parts. The analysis is conducted in three steps; gravity loading is applied first, followed by the application of operation loads, and, finally, the ground induced movement is imposed, holding one soil block fixed and imposing a displacement pattern in the external nodes of the second block. A fine mesh should be employed at the part of the pipeline where maximum stresses and strains are expected. Similarly, the finite element mesh for the soil should be more refined in the region near fault and coarser in the region away from the fault. The relative movement of the two blocks is considered to occur within a narrow zone of width w to avoid numerical problems.

Material models: The constitutive models should account for the elastic plastic behavior of both the pipeline and soil. Von Mises plasticity with isotropic hardening can be employed for describing pipe steel material, calibrated through a uniaxial stress strain curve from a tensile test. Furthermore, an elastic perfectly plastic Mohr Coulomb model can be considered for modeling soil behavior. This model is characterized by the soil cohesiveness c, the friction angle ϕ, the elastic modulus E, and the Poisson's ratio v. Furthermore, a contact algorithm should be employed to simulate the interface between the outer surface of the steel pipe and the surrounding soil, taking into account interface friction, and allowing separation of the pipe and the surrounding soil.

Analysis procedure and output: The analysis should follow a displacement controlled scheme, which increases gradually the ground displacement. At each increment of the nonlinear analysis, stresses and strains at the pipeline wall should be recorded. Furthermore, using a fine mesh at the critical pipeline portions, local buckling (wrinkling) formation and postbuckling deformation at the compression side of the pipeline wall can be simulated in an explicit manner.

Seismic Resistance of Steel Pipelines

Pipeline Performance Criterion and Limit States

In pipeline seismic design, the main target is pipeline integrity against loss of containment. One should notice that a severe seismic event may cause significant deformation of the pipeline, well beyond the elastic regime of the pipe steel material, so that traditional pipeline design based on allowable stress may not be applicable. Therefore, the corresponding performance criterion can be stated as pipeline may exhibit damage, but should maintain its water containment, so that it continues to fulfil its operational function after the seismic event.

Several limit states for continuous (welded) pipelines exist:

- Pipe wall fracture due to excessive tensile strain (base material and butt welded joints);
- Pipe wall local buckling due to excessive compressive strain;
- Pipeline overall buckling due to compressive loading; and
- Failure of welded lap joints (fracture or crushing) and pipe components.

In the course of a pipeline earthquake design procedure, the failure modes are quantified in terms of strain and deformation capacity, as described in the following subsection.

Maximum Tensile Strain Capacity

Exceedance of tensile strain capacity may cause fracture of pipeline wall. In the absence of serious defects or damage in the pipeline, the tensile capacity is governed mainly by the strength of the pipeline field welds, which usually are the weakest locations due to weld defects and stress/strain concentrations. Tensile strain limits of butt welds are experimentally determined through appropriate tension tests on strip specimens and on wide plates (Wang et al. 2010). Several standards and guidelines suggest a value of the ultimate tensile strain ε_{Tu} for butt welded water pipelines between 2 and 5%. The EN 1998 4 provisions adopt a value of 3% for tensile strain limit for seismic fault induced action on buried steel pipelines; however, it is not clear whether it is applicable to welded lap joints. The ALA (2005) limits for tensile strain are very similar, suggesting a limit strain equal to 2% for double welded lap joints. The PRCI (2004) suggests for the case of oil and gas pipelines a limit value within 2 4% for pressure integrity and a limit within 1 2% for normal operability. Finally, Annex C of the CSA Z662 pipeline design standard provides an equation for calculating tensile strain limit ε_{Tu} of pipeline girth welds, considering surface defects. These limit values for the maximum tensile strain ε_{Tu} refer to the macroscopic strain calculated from a stress analysis methodology, as described in the previous sections of this paper; this value of strain is quite different than the strain in the vicinity of the weld toe.

Local Buckling

Compressive ground induced strains also may occur due to axial compression and pipe bending deformation. When compressive strains exceed a certain limit, pipeline wall becomes structurally unstable and fails in the form of local buckling or wrinkling, as shown in Fig. 5(a) (Van Es et al. 2016; Vasilikis et al. 2016). Initially, despite the presence of those *wrinkles* or *buckles*, the pipeline may still fulfill its basic function (i.e., water transmission) provided that the steel material is adequately ductile (Gresnigt 1986). However, the buckled area is associated with significant strain concentrations and, in the case of repeated loading due to operation conditions (e.g., rather small variations of internal pressure or temperature), fatigue cracks may develop, imposing serious threat to the structural integrity of the pipeline (Dama et al. 2007; Pournara et al. 2015). Compressive strain limits for steel pipes depend primarily on the diameter:thickness ratio (D/t) and the level of internal pressure, and secondarily on the yield stress of steel material σ_y. Initial imperfections and residual stresses (as a result of the manufacturing process) also may have a significant effect on the critical compressive strain (Gresnigt and Karamanos 2009). The value of local buckling (ultimate compressive) strain ε_{Cu} can be estimated using the following design equation, initially proposed by Gresnigt (1986) and adopted by NEN 3650 and CSA Z662:

$$\varepsilon_{Cu} = 0.5\left(\frac{t}{D}\right) - 0.0025 + 3000\left(\frac{\sigma_h}{E}\right)^2 \tag{13}$$

where the hoop stress σ_h depends on the level of internal pressure p:

$$\sigma_h = \begin{cases} p(D/2t), & \text{if } p(D/2t) \le 0.4\sigma_y \\ 0.4\sigma_y, & \text{if } p(D/2t) > 0.4\sigma_y \end{cases} \tag{14}$$

Another equation for the ultimate buckling strain has been proposed by the DNV OS F101 standard (Det Norske Veritas 2012)

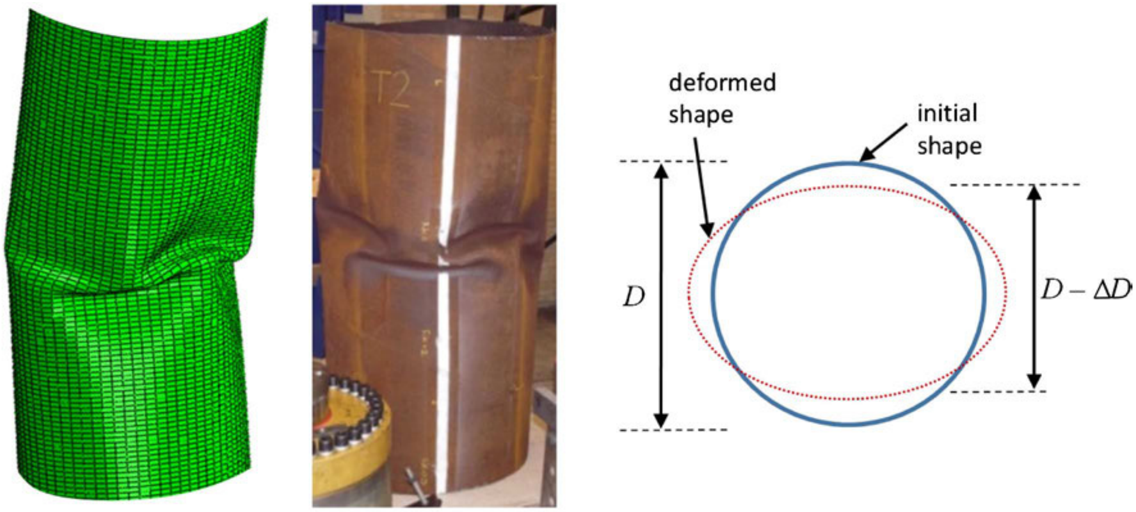

Fig. 5. (a) Local buckling of a spiral welded pipe with $D/t = 119$ due to excessive pipe wall compression, subjected to longitudinal bending; unpublished numerical simulation conducted at the University of Thessaly, and test conducted at Delft University of Technology, the Netherlands (data from Van Es et al. 2016; Vasilikis et al. 2016) (image by Spyros A. Karamanos); (b) definition of pipe cross sectional flattening

$$\varepsilon_{Cu} = 0.78\left(\frac{t}{D} - 0.01\right)\left(1 + 5.75\frac{p}{p_b}\right)\alpha_h^{\,1.5}\alpha_{gw} \quad (15)$$

where p_b = burst pressure; α_h = hardening factor that depends on the yield:tensile strength (Y/T) ratio; and α_{gw} = girth weld factor, given the fact that this equation has been proposed for girth welded pipes.

Beam Buckling

Under excessive quasi uniform compressive loading, the pipeline may buckle as a beam. The pipeline is very long with respect to its cross section, which means that it is very slender. Therefore the main resistance parameter against beam buckling is the lateral resistance offered by the surrounding soil. This implies that shallow trenches and/or backfills with loose materials may result in the activation of this failure mode. In general, beam buckling load is an increasing function of the cover depth and the stiffness of the backfill material. Hence, if a pipe is buried at a sufficient depth, it will develop local buckling before the occurrence of beam buckling. To design water pipelines against beam buckling, one may use the design tools for the design of high pressure high temperature oil and gas pipelines against beam buckling, referred to as *upheaval* or *thermal* buckling (Palmer and King 2008), or employ the nomographs proposed by Meyersohn (1991), also reported by O'Rourke (2003), which provide the critical cover depth of a buried pipeline. It should be noted that this failure mode is more likely to occur in oil and gas pipelines, in which significant axial compression may develop due to pressure and temperature. On the other hand, water pipelines may develop high compression in the case of a permanent ground induced action, mainly when loaded in the direction of the pipeline axis, and therefore this mode should be considered in the course of an earthquake design procedure.

Distortion of Pipeline Cross Section

To keep the pipeline operational, it is necessary to avoid significant distortions of the pipeline cross section. This is more likely to occur in low pressure thin walled pipelines, whereas internally pressurized pipelines exhibit less cross sectional distortion due to the stabilizing effect of internal pressure. This is a serviceability limit state, not related directly to failure and loss of containment, and a simple measure of cross sectional distortion is the nondimensional *flattening parameter* f defined in terms of the ratio of the maximum change of pipe diameter ΔD to the original diameter D [Fig. 5(b)]

$$f = \Delta D/D \quad (16)$$

Following Gresnigt (1986) and NEN 3650, a cross sectional flattening limit state is reached when the value of f becomes equal to 0.15.

Resistance of Pipeline Joints and Fittings

Welded lap pipe joints offer a simple and efficient way of connecting large diameter thin walled line pipes. The weld can be external, internal, or both. The eccentricity of the longitudinal stress path along the pipeline at this connection, together with the fillet type weld, may result in a reduction of pipe joint strength with respect to the strength of the line pipe itself. Furthermore, welded lap joint efficiency also depends on the ratio l/t, where l is the length of the curved portion of the female pipe (O'Rourke and Liu 2012). Limited work has been published on the response of those joints under severe structural loading. Mason et al. (2010b) experimentally investigated the tensile capacity of welded lap joints on small diameter [304.8 mm (12 in.)] pipes with D/t ratio equal to 50, significantly thicker than the pipes used for water transmission. They found that failure of the welded lap joints occurred at strains higher than 2%, which indicates that those joints were capable of sustaining inelastic deformation before failure. Moreover, the experimental testing on and finite element calculations for the compression strength of welded lap connections (Tsetseni and Karamanos 2007; Mason et al. 2010a) indicated that for pipes with D/t ratio equal to approximately 100, welded lap joint efficiency is close to 0.8 but decreases for pipes with higher values of D/t ratio. This efficiency value is significantly higher than the values suggested by the ASME B&PV (Boiler and Pressure Vessel) code, as noted by Smith (2006).

Karamanos et al. (2015b) examined the structural behavior of welded lap joints in large diameter pipes ($D/t = 150$, 240) subjected to bending in the presence of internal pressure using three dimensional nonlinear finite element models. They found that the principal mode of failure was local buckling at the joint area. Furthermore, the results indicated that upon occurrence of local buckling, local strains may increase very rapidly in several critical locations. More recently, McPherson et al. (2016) proposed a strengthening technique of welded lap joints using a steel outer bell, expanded in the pipe mill together with the bell of the parent pipe. Numerical calculations from three dimensional finite element models have shown that the outer bell provides extra strength to the welded lap joint and constitutes a promising and efficient joint strengthening solution for welded steel pipes constructed in geo hazard areas.

On the other hand, the behavior of pipe fittings (e.g., mitered elbows, pipe junctions) under severe structural loading has received less attention. Karamanos et al. (2016) studied the structural behav ior of mitered bends and addressed the issues of bending flexibility, stress intensity and local buckling failure. It is the authors' opinion that the mechanical behavior of pipe fittings subjected to severe ground induced actions and their effect on steel pipeline response constitutes an open issue that requires further investigation.

Use of Gasketed Joints in Seismic Areas

The use of gasketed joints in steel pipelines constructed in seismic zones has raised significant debate. Because of their ability to allow for a small amount of relative displacement and rotation between the two adjacent pipe segments, one argument supports the use of gas keted joints in seismic areas. More specifically, it has been argued that the relative motion of adjacent parts in gasketed joints may be able to accommodate ground induced pipeline actions in an efficient manner. It is the authors' opinion, however, that in the case of severe permanent ground deformations the capability of a *segmental* pipe line with gasketed joints to sustain significant tensile loading is questionable, mainly because the corresponding displacement at the joints may localize at one joint, resulting in excessive local rel ative displacement and loss of pipeline continuity.

Furthermore, the behavior of gasketed joints under severe bend ing loading is an open issue. A recent work on the behavior of gas keted joints on 6 in. diameter ductile iron pipes ($D/t = 21$) showed that those joints exhibited a substantial rotational capacity of 16 de grees (Wham and O'Rourke 2016). However, steel pipes employed in steel pipeline applications are much thinner than ductile iron pipes, and relative rotation due to severe bending will cause high local strains and deformations that may damage the pipe and the gasket, leading to loss of containment. A dedicated investigation that combines experimental and numerical work is necessary in or der to determine reliable deformation limits for gasketed joints sub jected to bending in large diameter steel pipes.

On the other hand, it is expected that gasketed joints, properly designed, are capable of accommodating seismic transient effects, and therefore can be employed in seismic zones where severe per manent ground induced actions are not expected. Following the provisions of ALA Guidelines (2005), the displacement Δ_{joint} that the gasketed joint should be able to sustain from transient seismic action is equal to

$$\Delta_{\text{joint}} = 7L_p\varepsilon_g + 0.25 \text{ in.} \tag{17}$$

where ε_g = ground strain of Eq. (1); and L_p = length of a pipe segment. In Eq. (17), the extra value of 0.25 in. is considered as a factor of safety, and a factor equal to 7 is introduced to account for the uncertainly associated with the distribution of axial dis placement in a segmental pipeline under tensile loading; because the corresponding expansion may not be equally distributed in all gasketed joints, the deformation at one joint may localize, so that the two pipeline parts are separated, leading to loss of contain ment. This design procedure is described in the "Design Example" section. Finally, O'Rourke et al. (2015) conducted a fragility analy sis of such joints under seismic wave loading.

Mitigation Measures against Seismic Actions

Several measures can be employed to mitigate seismic damage to pipelines. The most obvious action to minimize earthquake ef fects is the modification of pipeline alignment to avoid seismic and geohazard areas (pipeline rerouting). However, in the majority of cases, this may not be possible; therefore specific mitigation mea sures should be adopted to minimize ground induced strains in the buried pipeline. Specific measures include:

- The increase of pipeline wall thickness increases pipeline strength against seismic action. Both buckling and tensile resis tance of the pipeline wall increase with increasing thickness.
- The use of higher grade line pipe material increases pipeline strength. However, one may be cautious for the reduced ductility of high strength steel, usually expressed through the yield:ten sile strength ratio (Y/T); permanent ground actions are applied through a displacement controlled scheme, and in such a case material ductility and deformation capacity may be more impor tant than strength.
- In areas where significant permanent ground deformations are expected, the designer may consider isolating the pipeline from the ground movements, using either an above ground pipeline section, appropriately supported in the ground, or a tunnel around the pipeline, so that the pipeline does interact with the surrounding soil.
- In landslide areas, it may be possible to improve ground con ditions using a slope drainage system so that the risk of slope instability is reduced.
- In fault crossings, stiff soil conditions introduce higher stresses and strains in the pipeline. Therefore the use of soft backfill soil will result in reduced stresses and strains within the pipeline. However, a soft cover may reduce its resistance in global buck ling, and therefore such a solution should be used cautiously.
- In strike slip faults, the crossing angle should be such that the pi peline is in tension and not in compression. Based on recent finite element results (Vazouras et al. 2015), a crossing angle equal to 10 20° appears to be an optimum angle for strike slip faults.
- In fault crossing, the use of flexible components (e.g., elbows), may not be recommended within the fault zone. Nevertheless, in fault crossings associated with significant pipeline tension, using elbows at an appropriate distance from the discontinuity area may result in a reduction of axial stretching and the corre sponding strains; the distance depends on elbow geometry, soil properties, and the direction of the fault.
- Where possible, reverse vertical faults (thrust faults) should be avoided because they result in high compressive stresses, which may cause buckling of thin walled steel pipes.
- Specialized expansion joints and/or deflectable joints can be used as mitigation devices to reduce axial stretching of the pipe line in permanent ground motion areas.

Design Example

A buried steel pipeline was considered for a seismic zone. Seismic activity consisted of transient seismic wave action, characterized by peak ground acceleration and velocity equal to 0.30 g and

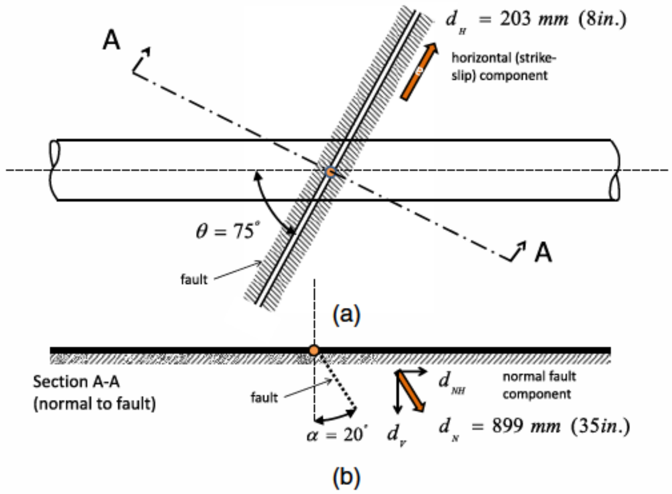

Fig. 6. Pipeline crossing a seismic fault in a seismic zone: (a) plan view of pipeline crossing; (b) section A A

76.2 cm/s, respectively, and by a seismic fault crossing the pipeline with left lateral strike slip offset of 203 mm together with normal offset along the fault surface of 899 mm, as shown in Fig. 6. The pipeline had diameter and thickness equal to 1,524 and 8.1 mm (60 and 0.319 in.), respectively, with material X42 steel grade ($\sigma = 290$ MPa), and was buried at a depth equal to one pipe diameter D with respect to the top side of the pipe. Soil conditions were cohesionless, with the following properties: density $\gamma = 17.4\ \mathrm{kN/m^3}$, friction angle $\varphi = 34°$, and lateral earth pressure coefficient K_0 assumed equal to 0.5. For the sake of simplicity, the pipeline alignment was considered to be straight, without pipe bends. The following sections outline pipeline behavior and design for both seismic wave action and fault movement.

Pipeline Joint Configuration

The steel pipeline will be welded (continuous), with welded lap joints in the fault crossing area, and segmental, with gasketed joints away from this area. Fig. 7 shows the configuration of the gasketed joint; the welded lap joints are considered double welded (inside and outside weld) to maximize their strength.

A welded (continuous) pipeline should be considered in the analysis of permanent ground induced fault action, whereas the analysis of seismic wave action should refer to a segmental pipeline. The former analysis should also determine the length of welded pipeline segment. These two analyses are briefly de scribed below.

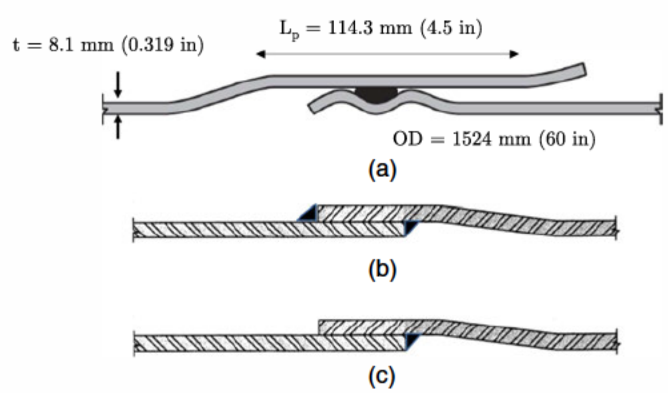

Fig. 7. (a) Configuration of gasketed joint used in the pipeline; (b) double welded lap joints; (c) internally welded lap joints

Seismic Wave Action

Seismic wave action on the pipeline was calculated from Eq. (17) in terms of the relative displacement Δ_S in a gasketed joint, assuming a line pipe length equal to 12.1 m (40 ft) and apparent velocity equal to 3,050 m/s (10,000 ft/s) as follows:

$$\Delta_S = 7L_p\varepsilon_g = 7L_p\frac{\mathrm{PGV}}{C} = 21\ \mathrm{mm}\,(0.84\ \mathrm{in.}) \tag{18}$$

and the total seismic displacement is equal to

$$\Delta_{\mathrm{joint}} = \Delta_S + 0.25\ \mathrm{in.} = 27.7\ \mathrm{mm}\,(1.09\ \mathrm{in.}) \tag{19}$$

This displacement of 27.7 mm can be sustained by the gasketed joint shown in Fig. 7.

Fault Crossing Analysis

Based on Fig. 6, the three components of pipeline action with re spect to the pipeline (axial, horizontal transverse, vertical trans verse) were computed from the following geometric equations:

$$\delta_{Ha} = d_H\cos\theta + d_N\sin\alpha\sin\theta \tag{20}$$

$$\delta_{HT} = d_H\sin\theta - d_N\sin\alpha\cos\theta \tag{21}$$

$$\delta_V = d_N\cos\alpha \tag{22}$$

The values of δ_{Ha}, δ_{HT} and δ_V are equal to 346.3 mm (13.63 in.), 117.6 mm (4.63 in.), and 835.4 mm (32.89 in.), and correspond to the values of $\mathrm{PGD}_{FH}\cos\theta$, $\mathrm{PGD}_{FH}\sin\theta$, and PGD_{FV}, respectively, in Eqs. (5) (8).

Subsequently, the axial (membrane) strain was computed from Eq. (7), with values of lengths L_H, L_V, and L'_V equal to 15.75 m, 14.67 m, and 6.06 m, respectively (Fig. 2). These values were found by applying the methodology proposed by Sarvanis and Karamanos (2016). Subsequently, the bending strain in the vertical plane also was computed from Eq. (8). The maximum strain value was 3.46%, shown in Table 1. This strain is within the 2 5% range.

In addition to the above analytical calculations, this fault cross ing was analyzed using finite element models (Level 1), which em ployed special purpose elements for the pipe (elbow elements) and nonlinear springs for the soil. Spring constants were determined according to ALA (2005) guidelines and are shown in Table 2. Fig. 8 shows the distribution of axial strains; the maximum strain was equal to 3.09% (Table 1), which is quite close to the value computed using the analytical expressions of Eqs. (7) and (8) yet somewhat lower, which is beneficial for the pipeline. This value is very close to the limit specified by EN 1998 4 (3%) and is accept able. However, according to ALA (2005) guidelines it may not be sustained by a welded lap joint because it exceeds 2%. In such a

Table 1. Comparison between Finite Element Model and Design Equations

Parameter	Maximum tensile strain (%)
Analytical	3.46
Finite element model	3.09

Table 2. Maximum Soil Resistance and Corresponding Displacements according to ALA (2005)

Soil springs according to ALA (2005)	Force per unit length of pipe (kN/m)	Corresponding displacement (m)
Axial	52.9	0.005
Horizontal	544	0.090
Vertical: uplift	90	0.015
Vertical: bearing	2,560	0.150

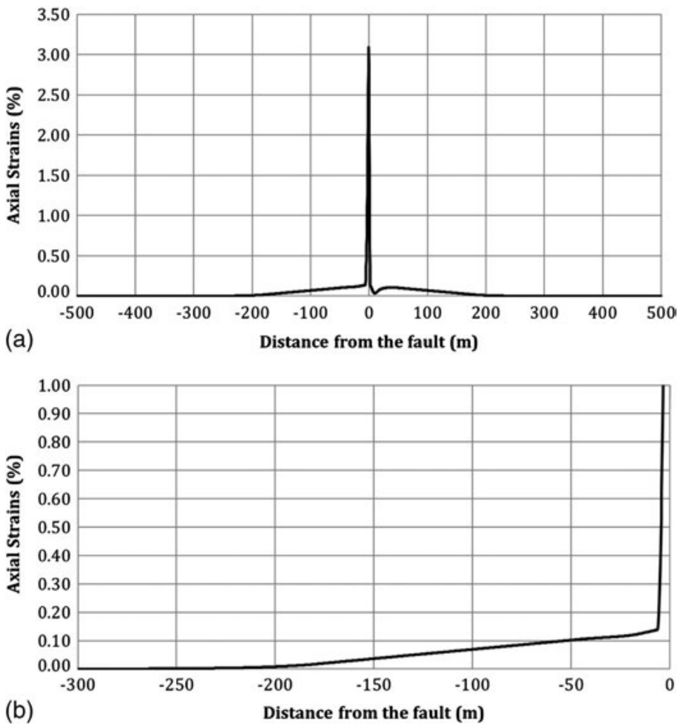

Fig. 8. Axial strains along the pipeline at the fault crossing area: (a) strain distribution along a long pipe segment; (b) strain distribution up to 300 m from the fault

case, mitigation measures are necessary, such as pipeline realign ment or the use of a softer backfill, in an attempt to reduce ground induced tensile strain. Furthermore, strengthening of the welded lap joints may increase pipeline resilience. In any case, it is authors' opinion that the development of reliable tensile strain limits for welded lap joints is necessary and should be a research priority.

The numerical results also show that the high strains are devel oped in a small length of 10 m around the fault, whereas outside this zone the strain level does not exceed 0.2%. More specifically, pipeline stretching decays rapidly outside the fault zone and be comes negligible at a distance of approximately 250 m from the fault plane on either side of the fault. It is the authors' opinion that within this length, the pipeline should be welded (total length of welded pipeline equal to 500 m), given also the uncertainty on the exact location of the fault. Outside this length, segmental (gasketed) joints can be employed.

Summary and Conclusions

Seismic design of buried steel water pipelines is a topic of signifi cant importance for safeguarding the structural integrity of pipe lines constructed in seismic zones. However, current pipeline design standards contain limited information on seismic design. The ALA (2005) guidelines, together with the Indian NICEE rec ommendations (2007), constitute documents on this subject that can be used for design purposes, whereas the PRCI (2004) guide lines refer mainly to hydrocarbon pipelines.

Soil pipe interaction is the key issue for determining ground induced strains on the pipe wall. For the case of permanent ground induced actions, the designer may use a finite element model for efficient stress analysis of the pipeline. However, analyti cal expressions can be used to obtain reasonable estimates of ground induced strains in the pipeline. Furthermore, this paper de scribed the main issues related to the mechanical behavior and pipe resistance of buried thin walled welded steel pipelines, referring to the relevant failure modes. It is the authors' opinion that additional research is necessary to determine the strength and deformation capacity of pipeline joints and fittings under axial and bending loading.

The design framework was applied in a specific case study that involved both permanent and transient seismic actions, and showed that an appropriate combination of welded lap and gasketed joints may offer a good solution for buried steel pipelines constructed in seismic zones.

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