Seismic design of buried steel water pipelines

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ABSTRACT

The present paper provides an overview of available tools and provisions for the structural analysis and design of buried welded (continuous) steel water pipelines in seismic areas, subjected to earthquake action. Both transient and permanent ground actions (coming from tectonic faults, landslides and liquefaction-induced lateral spreading) are considered. Specific issues are discussed on the modelling of the interacting pipeline-soil system using either simple analytical models or nonlinear finite elements, and their main advantages and disadvantages are pin-pointed. Subsequently, the resistance of buried pipelines is discussed, with emphasis on possible failure modes. Finally, possible mitigations measures for reducing seismic effects are discussed, for the safe operation of steel water pipelines against seismic hazard.

INTRODUCTION

There is an increasing concern for the structural performance of steel water pipelines in geohazard areas. In the particular case of earthquake action, the main purpose of pipeline operators is to minimize seismic risk on the pipeline, safeguarding the unhindered flow of water resources, following an earthquake event. Towards this purpose, the structural damage of the steel pipe should be minimized, in order to maintain the structural integrity of the pipeline and prevent leakage.

Seismic hazards for buried steel pipelines can be classified into two main categories: (a) transient actions, associated with wave propagation shaking phenomena and (b) permanent ground-induced deformations, such as seismic faults, landslides, subsidence settlements, and liquefaction-induced lateral spreading. Buried pipelines have sustained significant damage in past earthquakes, as noted by O’Rourke (2003). These damages have been attributed to both transient and permanent ground deformations (EERI, 1999; Liang and Sun, 2000). It is noted that damage due to permanent ground-induced actions typically occurs in specific areas of severe ground failure, and it is associated with high damage rates (O’Rourke, 2003), whereas shaking damages occur over significant larger areas, but they are associated with lower damage rates.

The vast majority of research work reported in the seismic analysis and design of steel pipelines has been motivated by the safety of hydrocarbon (oil and gas) pipelines. The present paper does not intend to provide a complete literature review on this subject. For transient actions, the reader is referred to the paper by Kouretzis et al. (2006) for an extensive literature review, whereas the recent paper by Vazouras et al. (2012) offers a good summary of previous works on permanent ground deformations on buried pipelines. Significant research on permanent ground-induced deformation on buried steel pipelines is being performed in the course of the GIPIPE project (Karamanos *et al*., 2013), where large-scale experiments are being conducted, supported by extensive numerical simulations. It is worth noting that water pipeline design standards, such as AWWA M11, do not contain provisions for seismic design.

Nevertheless, there exist some important differences between hydrocarbon and water pipelines, so that direct application of design guidelines and tools from hydrocarbon to water pipelines may not be appropriate. Steel water pipelines differentiate from hydrocarbon steel pipelines because they:

- are considerably thinner, with much higher values of D/t ratio
- are made of lower steel grade; X42 or X46 are usual grades for water steel pipes, whereas hydrocarbon pipelines use X70 grade or higher in onshore pipeline applications.
- have different type of joints; instead of butt-welded joints, used almost exclusively in hydrocarbon pipelines, continuous water pipelines are constructed with welded-slip lap joints.
- operate under lower pressure levels; this may not be necessarily beneficial, given the fact that, sometimes, internal pressure can prevent cross-sectional distortion, thus increasing pipeline deformation capacity.
- contain special components (e.g. elbows and junctions) with significantly different geometry (configuration) than in oil & gas pipelines.

In pipeline design against earthquake action, the main requirement pipeline actions S should less than the corresponding pipeline resistance R. The present paper offers an overview of seismic analysis and design of buried welded steel pipelines for water transmission and distribution, 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 recommendations in North America and Europe, the paper refers to seismic actions, due to both transient and permanent ground deformations. Subsequently, issues related to pipeline resistance are presented, with direct reference to possible failure modes. Finally, measures for mitigation of seismic effects on pipelines are briefly discussed for the safe operation of steel water pipelines in areas of high-seismicity.

EXISTING STANDARDS AND RECOMMENDATIONS FOR PIPELINE SEISMIC DESIGN

The ASCE (1984) Guidelines is the first document that attempted transferring and adjustment of existing knowledge and design tools of earthquake engineering into the analysis and design of pipelines, representing mainly the work on this subject by N. M. Newmark, W. J. Hall and their associates at the University of Illinois (e.g. Newmark, 1967; Newmark and Hall, 1975). Apparently, this document has constituted the basis for the ALA (2005) design guidelines, which is the document with the most complete set of provisions for this subject. Some of those provisions will be used in the present paper. The above work also constitutes the basis for the more recent Indian NICEE Guidelines (2007) for seismic design of buried pipelines.

In ASME B31.4 and ASME B31.8, widely used for oil and gas pipeline design respectively, it is specified that seismic loading should be taken into account as accidental (environmental) loading. However, they do not contain information on how earthquake action on the pipeline should be computed. Similarly, Canadian standard CSA Z662 specifies slope movements, fault movements and seismic-related earth movements as additional loading that should be considered in the course of a pipeline stress design, but does not contain any further information on the quantification of those seismic actions.

European standard EN 1594 has been widely employed for the general design of buried gas pipelines. Annexes D and E of this standard refer to landslide and high-seismicity areas respectively; in both Annexes, it is suggested to analyze the pipeline against these geohazards and some mitigation measures are proposed. Similarly, EN 16416 standard, also known as ISO 13623 standard, contains subsection 6.3.3.3 with general information and suggestions on earthquake design. European standard EN 1998-4, also provides guidance for the seismic design of buried pipelines. The standard is primarily dedicated with the seismic design of liquid storage tanks, whereas limited information on buried pipelines is contained in Chapter 6 and Annex B. In addition, EN 1998-4 is intended to cover all possible materials (steel, concrete, plastic), and therefore, it may not be a standard suitable for the seismic design of buried steel pipelines. Finally, among national pipeline design standards, one may highlight the Dutch standard NEN 3650; despite the fact that seismic action is not an issue in The Netherlands, this standard contains valuable information for ground-induced action on pipelines and for soil-pipe interaction in settlement areas.

SEISMIC ACTIONS IN CONTINUOUS BURIED PIPELINES

There exist two main sources of ground-induced seismic deformations on buried pipelines, namely the transient actions and the permanent deformations. Transient actions are caused by wave propagation within the soil, whereas permanent ground deformations are due to fault movements, landslide activation and liquefaction-induced lateral spreading. Herein, the effects of ground-induced seismic actions on continuous steel buried pipelines are examined. Those are welded pipelines, using mainly welded-slip joints. Butt-welded connections are used only in few instances.

Transient action

Transient action, referred to as “wave propagation hazard”, is characterized by peak ground acceleration and velocity, as well as the appropriate propagation velocity, and is caused by ground shaking due to travelling body and surface seismic waves. Body waves [compressional and shear] propagating through the three-dimensional ground, are generated by seismic faulting at the seismic source. Surface waves [Love and Rayleigh], travelling along the ground surface are generated by the boundary condition imposed by ground surface to body waves at the ground surface.

The analysis of wave action on a buried pipeline is a rather complex problem requiring wave propagation analysis on the three-dimensional soil-pipe system, accounting for their interface. Alternatively, one may use simplified method developed by Newmark (1967) to estimate soil strain and curvature due to a traveling wave of constant shape, in terms of peak ground motion parameters. In particular, the maximum ground strain ε_g in the direction of wave propagation can be computed as follows:

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

where PGV is the maximum horizontal ground velocity in the direction of wave propagation and C is the apparent propagation 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 is the ultimate frictional force of soil per unit pipe length acting on the pipe in the axial direction and λ is the seismic wavelength in soil at pipe location. Similarly, 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 expression:

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

where PGA is the maximum ground acceleration perpendicular to the direction of wave propagation. The peak ground motion parameters PGV and PGA can be obtained from earthquake records in the area of interest or from relevant seismic maps.

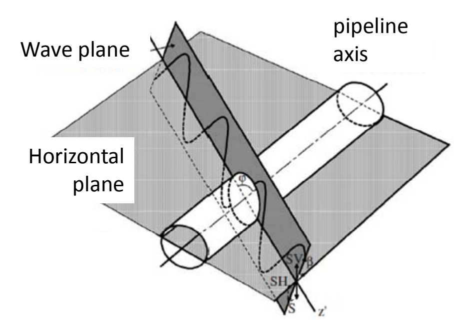

Figure 1: Shear wave analysis, oriented randomly with respect to the pipeline.

Theoretically, if the direction of interest is not parallel to the direction of wave propagation, the value of C in the above relations for ground strain and curvature need to be adjusted, as shown in Figure 1. However, in the course of a pipeline seismic design procedure, the direction of the propagating wave is not known a priori and, therefore, the use of the nominal wave velocity may be used for simplicity in the denominator of those equations. Finally, it is assumed that entire soil deformation is transmitted to the pipeline, so that (1) and (4) can be used for estimating the axial strain and the bending curvature in the buried pipeline. This is a conservative assumption, but in lieu of a detailed analysis, is can be used for design purposes.

Permanent ground deformation – analytical methods

A significant number of seismic damages to steel pipelines have been caused by permanent ground deformations such as fault movements, landslides and liquefaction-induced lateral spread. Permanent ground deformations are applied on the pipeline in a quasi-static manner, and they are not necessarily associated with high seismic intensity; nevertheless, the pipeline may be seriously damaged.

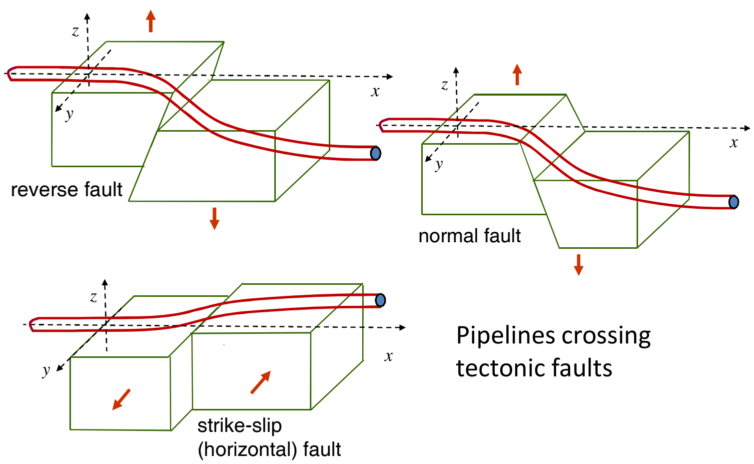

Figure 2: Schematic representation of pipeline configuration crossing tectonic faults.

Tectonic Faults

An active tectonic fault is a discontinuity between two portions of the bedrock, along which relative motion of the two portions can 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), vertical (normal or reverse fault) or at an oblique direction (oblique fault), as shown in Figure 2. It is possible to estimate fault displacement PGD_F in terms of earthquake moment magnitude using empirical relations, e.g. Wells and Coppersmith (1994).

Subsequently, the axial strain induced by the fault movement in the pipeline wall can be computed analytically, following the procedure in Kennedy et al. (1977). For the case of horizontal faults (Figure 3), using the horizontal ground-induced displacement PGD_{FH} the maximum axial strain is:

$$\varepsilon = \frac{PGD_{FH}}{L_H}\cos\theta + \left(\frac{PGD_{FH}}{3L_H}\sin\theta\right)^2 \qquad (5)$$

where θ is the angle between the fault plane and the pipeline axis, L_H is the distance between the fault and the "anchor point", estimated by the following expression:

$$L_H = \sqrt{(F_Y / k_H) \sin\theta} \tag{6}$$

where k_H is the horizontal soil stiffness and F_Y is the plastic axial force. In the case of an oblique fault, with simultaneous fault movement PGD_{FV} in the vertical direction, one may write the following equation for the axial strain in the pipeline,

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

where L_V is the distance between the fault and the "anchor point" in the vertical plane, estimated from equation (6) using the vertical soil stiffness k_V.

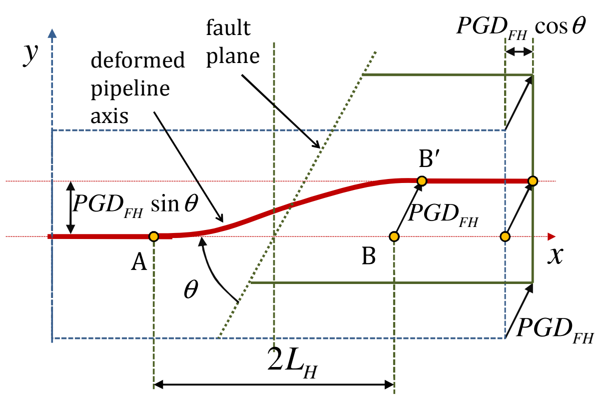

Figure 3: Pipeline deformation crossing a horizontal fault at angle θ.

Landslides

A landslide is associated with massive ground movement because of soil slope instability (Figure 4a). Gravity is the primary driving force, but an earthquake event may trigger a landslide to occur. Various empirical methodologies have been proposed to determine the occurrence a landslide in terms of the distance from the epicentre and the magnitude of the earthquake event. Moreover, the expected landslide movement PGD_S is required to quantify the effects of landslide on a pipeline, and can be computed by several analytical expressions, e.g. Jibson (1994). For permanent ground-induced action in the longitudinal direction due to landslide, the pipeline should be designed for an axial force F, which is the minimum of F_1 and F_2, given by the following equations:

$$F_1 = \sqrt{EAt_u\left(PGD_S\right)} \tag{8}$$

and

$$F_2 = (t_u L_S)/2 \tag{9}$$

In the above expressions, t_u is the ultimate frictional force of soil per unit pipe length acting on the pipe in axial direction and L_S is the length of pipe in soil mass undergoing movement. According to ALA (2005), the value of L_S may range between 100 and 250 meters.

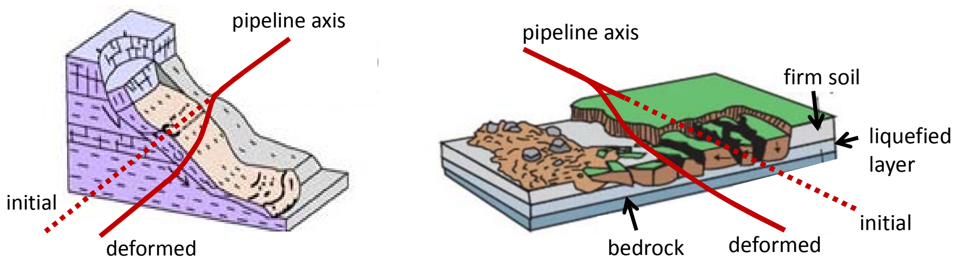

Figure 4: Schematic representation of pipeline configuration in the boundary of landslide (left) and liquefaction-induced lateral spreading (right) [source: USGS; http://pubs.usgs.gov/].

For transverse permanent ground-induced action due to landslide, the bending strain in the pipeline can be estimated by the following expression, assuming a cosine function of the pipe deformation:

$$\varepsilon_b = \frac{\pi^2 D (PGD_S)}{W^2} \tag{10}$$

where W is the width of the landslide zone, which may range between 150 and 300 meters according to ALA (2005). Alternatively, assuming a beam with both ends fixed and a uniform lateral load p_u one readily obtains for the bending strain:

$$\varepsilon_b = \frac{p_u W^2}{3\pi E t D^2} \tag{11}$$

Lateral spreading

Lateral spreading is a consequence of liquefaction in a sandy soil layer; the soil to loose its shear strength, resulting in the lateral movement (flow) of the liquefied soil, primarily in the horizontal direction (Figure 4b). In liquefaction-induced lateral spreading, if the pipeline is contained in the liquefied layer, buoyancy should be taken into account, together with the horizontal ground movement imposed to the pipeline. To estimate permanent ground displacement due to liquefaction PGD_L, several expression have been proposed (e.g. Bardet et al., 2002). For longitudinal action, the corresponding maximum axial force in the pipeline can be calculated through equations (8)-(9), whereas for transverse lateral-spreading action, the maximum bending strain can be computed from equations (10)-(11).

Permanent ground deformation – finite element models

As a more rigorous alternative to design equations, it is possible to employ the finite element method to model the effects of ground-induced actions on a buried pipeline.

This analysis requires some computational effort and expertise, but offers an advanced tool for determining stresses and strains within the pipeline wall with significantly increased accuracy with respect to the analytical formulae described above. There exist two levels of finite element modeling, briefly described below. 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

In this type of analysis, the pipe is modelled with beam-type one-dimensional finite elements. This numerical methodology has been mainly employed for simulating permanent ground-induced actions on pipelines, but wave effects can also be modelled. The mesh near discontinuities (e.g. fault plane) should be fine enough, so that gradients of stress and strains are accurately simulated (Figure 5a).

Type of finite elements: The use of regular beam elements for the pipeline model is not recommended, because they cannot account for pressure. Instead, "pipe elements" are preferable, which account for hoop stress and strain due to pressure. Furthermore, the use of "pipe elements" with the capability 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. Alternatively, it is possible to use pipe elements with circular cross-section, and account for ovalization effects at pipe bends through appropriate flexibility factors, and stress intensity factors.

Pipe and soil modelling: Pipe material should be modelled as elastic-plastic, considering strain-hardening. The ground surrounding the pipeline should be modelled by nonlinear springs (Figure 5a), attached on the pipe nodes and directed in the transverse directions (with stiffness k_V and k_H in the vertical and lateral direction respectively) and axially (k_{ax}). The springs should account possible slip of the pipe through the soil and expressions for their stiffness are offered in ALA (2005), based on the type of soil. Alternative equations for those springs are offered in NEN 3650 standard.

Analysis procedure and output: To conduct pipeline analysis subjected to permanent ground deformation, appropriate displacements should be applied to the ends of the soil springs. The analysis is conducted in three steps: (a) gravity, (b) operational loading (pressure and temperature) and (c) PGD application. The analysis output consists of stress resultants in pipeline cross-sections, as well as the stresses and strains in the longitudinal direction. The user should be cautioned that if the finite elements are not capable of describing accurately cross-sectional distortion these stresses and strains may be quite different than the real stresses and strains in the pipeline wall, especially when the pipe wall begins to wrinkle due to local buckling. Consideration of local stresses due to pipe wall wrinkling locations requires a more detailed analysis, with the use of shell elements for modelling the pipe.

Level 2: three-dimensional finite element analysis

The use of three-dimensional finite elements offers a rigorous numerical tool to simulate buried pipeline behavior under PGD, but requires computational expertise. Such a model can describe 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, as well as the interaction between the pipe and the soil.

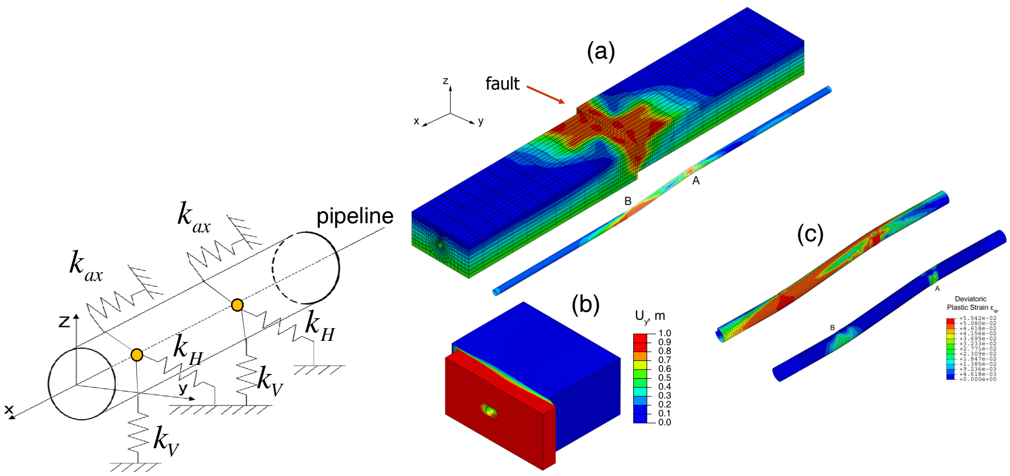

Figure 5: Level 1 of pipeline modelling; pipe (beam-type) finite elements and soil springs attached to pipeline nodes in the three principal directions (left); Level 2 of pipeline modelling; shell elements and solid elements (right) [Vazouras *et al*., 2010].

The basic idea is the consideration of an elongated prismatic model where the steel pipeline is embedded in the soil, as shown in Figure 5b for the case of a strike-slip fault. Shell elements are employed for modeling the pipeline cylinder, whereas 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, an 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.

Elastic-plastic material behavior is considered for both the pipeline and soil. Pipe material can be described with von Mises plasticity with isotropic hardening, calibrated through an appropriate uniaxial stress-strain curve from a tensile test. An elastic-perfectly plastic Mohr-Coulomb model can considered for the soil behavior, characterized by the soil cohesiveness c, the friction angle ϕ, the elastic modulus E, and Poisson's ratio v. A contact algorithm should be considered 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.

The analysis proceeds using a displacement-controlled scheme, which increases gradually the ground displacement. At each increment of the nonlinear analysis, stresses and strains at the pipeline wall are recorded. Furthermore, using a fine mesh at the critical pipeline portions, local buckling (wrinkling) formation and post-buckling deformation at the compression side of the pipeline wall can be simulated in a rigorous manner.

SEISMIC RESISTANCE OF STEEL PIPELINES

There exist 4 main failure modes for continuous (welded) pipelines, namely:

- 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
- Pipeline welded-slip joint failure (fracture or crushing)

The failure modes are quantified in terms of strain and deformation capacity.

Maximum tensile strain capacity

Tensile strain capacity is directly related to pipe wall fracture. In the absence of serious defects and damage of the pipeline, tensile capacity is controlled mainly by the strength of the pipeline field lap or butt welds, which are usually the weakest locations due to weld defects and stress/strain concentrations. Tensile strain limits are experimentally determined through appropriate tension tests on strip specimens and in wide plates (Wang *et al*., 2010). It is the authors' suggestion that the value of the ultimate tensile strain ε_{Tu} for butt-welded water pipelines should vary between 2% and 5%. It is noted that the value of 3% is adopted by the EN 1998-4 provisions for seismic-fault-induced action on buried steel pipelines and by the seismic provisions of ASCE MOP 119 for buried water steel pipelines. An equation for determining tensile strain limit ε_{Tu} of pipeline girth welds is provided by CSA Z662 pipeline design standard, Annex C, considering surface-breaking defects, and provides results within this range. One should note that the above limit values for the maximum tensile strain ε_{Tu} is the "macroscopic" strain calculated from a stress analysis methodology as described in the previous sections of this paper. It is quite different than the strain in the vicinity of the girth weld toe.

Local buckling

Under ground-induced actions, compressive strains may also occur due to pipe bending deformation. When compressive strains exceed a certain limit, pipeline wall exhibits structural instability in the form of local buckling or wrinkling, as shown in Figure 6. In the presence of those "wrinkles" or "buckles", the pipeline may still fulfill its basic function (i.e. water transportation), 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 (e.g. due to rather small variations of internal pressure or temperature), fatigue cracks may develop, imposing serious threat for the structural integrity of the pipeline (Dama *et al*., 2007). Compressive strain limits for steel pipes depend primarily on the diameter-to-thickness ratio (D/t), the presence of internal or external pressure, and secondarily on the yield stress of steel material σ_y. Initial imperfections and residual stresses (as a result of the manufacturing process) have a significant effect on buckling strain (Gresnigt and Karamanos, 2009). The local buckling (ultimate) compressive strain ε_{Cu} can be estimated using the following design equation, initially proposed by Gresnigt (1986), adopted by NEN 3650 and CSA Z662:

$$\varepsilon_{Cu} = 0.5\left(\frac{t}{D}\right) - 0.0025 + 3000\left(\frac{\sigma_h}{E}\right)^2 \qquad (12)$$

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

$$\sigma_h = \begin{cases} p(D/2t), & if \quad p(D/2t) \le 0.4\sigma_y \\ 0.4\sigma_y, & if \quad p(D/2t) > 0.4\sigma_y \end{cases} \qquad (13)$$

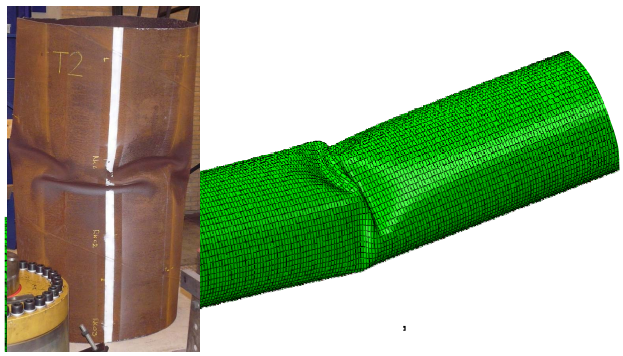

Figure 6: Local buckling of spiral welded pipe with $D/t = 119$ due to excessive pipe wall compression, subjected to longitudinal bending [Vasilikis *et al*., 2014].

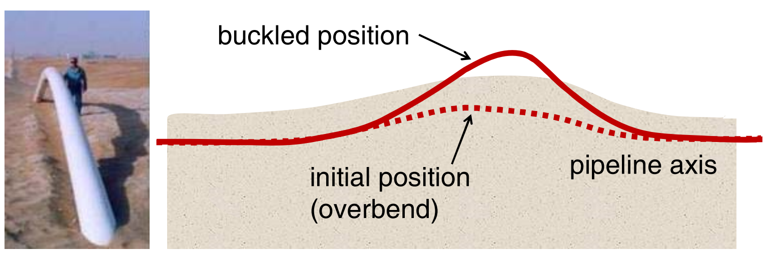

Figure 7: Beam buckling of buried pipeline due to excessive axial loading.

<u>*Beam buckling*</u>
Under excessive quasi-uniform compressive loading, the pipeline may buckle as a beam (Figure 7). The pipeline is very slender and the main resistance parameter against beam buckling is the lateral resistance offered by the surrounding soil. This means 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 beam buckling. To design water pipelines against beam buckling, one may use the design tools for the design of high pressure – high temperature hydrocarbon pipelines against beam-buckling, referred to as "upheaval" or "thermal" buckling (Palmer and King, 2008). Instead of such a detailed analysis, one may employ the nomographs proposed by Meyersohn (1991), also reported by O'Rourke (2003), which provide the critical cover depth of a buried pipeline. These nomographs

have been obtained setting the lowest beam buckling stress equal to stress that causes local buckling to determine cover depth so that beam buckling occurs before local buckling.

Distortion of pipeline cross-section
To maintain the pipeline operational, it is necessary to avoid significant distortions of the pipeline cross-section. This is more pronounced in low-pressure pipelines. A simple and efficient measure of cross-sectional distortion is non-dimensional "flattening parameter" f defined in terms of the change of pipe diameter ΔD as follows:

$$f = \Delta D / D \tag{14}$$

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 welded-slip pipeline joints
Welded-slip pipe joints offer a simple and efficient way for connecting large-diameter thin-walled pipelines. The weld can be external, internal or at both sides. Nevertheless, the eccentricity of 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. Tensile capacity of welded-slip joints has been investigated by Mason *et al*. (2010). The investigation was mainly experimental in 12-inch pipes with D/t ratio equal to 50, significantly thicker than the pipes used for water transmission. It was found that failure of the welded-slip joints occurred at strains higher than 2%, which indicates that those joints are capable of sustaining inelastic deformation before failure, and an allowable strain of 1% - 1.5% has been suggested. Furthermore, the experimental testing and finite element calculations on the compression strength of welded-slip connections (Tsetseni and Karamanos, 2007; Mason *et al*., 2010), has shown that for pipes with D/t ratio equal to about 100, the joint efficiency is close to 0.8, and reduces for pipes with higher values of the D/t ratio. This efficiency value is less than the values suggested by the ASME B&PV code, also noted by Smith (2006). Finally, is should be noted that there exists no information on the mechanical behavior of those joints under bending action. Both the ultimate moment and the rotational capacity of those joints is an open issue that has not been investigated yet.

MITIGATION MEASURES AGAINST SEISMIC ACTIONS

Several measures can be employed to mitigate seismic damage to pipelines. The first action and most obvious action is the modification of pipeline alignment (pipeline re-routing) to avoid geo-hazard areas. However, in several cases, this may not be possible; therefore, other mitigation measures should be adopted. More specifically:

- The increase of pipeline wall thickness increases pipeline strength against seismic action. Both buckling and tensile resistance of the pipeline wall are nearly proportional to 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; permanent ground

actions are applied through a displacement-controlled scheme and – in such a case – material ductility and deformation capacity may be more important than strength.

- In areas where significant permanent ground-induced actions are expected, the designer may consider to isolate the pipeline from ground movements, using an above-ground section appropriately supported in the ground.
- In landslide areas, it may be possible to improve ground conditions, and reduce the amount of ground movement, especially.
- The use of flexible joints, capable of accommodating imposed expansion/contraction or rotation at appropriate locations, can be beneficial for the pipeline, reducing the induced strains, especially axial stretching.
- In fault crossings, stiff soil conditions introduce higher stresses and strains in the pipeline. Therefore, the use of soft backfill soil would result in reduced stresses and strains within the pipeline. However, a soft cover may reduce its resistance in global buckling, and therefore, such a solution may be used cautiously.
- In strike-slip faults, the crossing angle should be such that the pipeline is in tension and not in compression. Based on recent finite element results, a crossing angle equal to 10-20 degrees appears to be an optimum angle.
- In fault crossing, the use of flexible components (e.g. elbows), at a distance from the discontinuity area, would result in a reduction of axial stretching and the corresponding strains.
- Where possible, reverse vertical faults should be avoided because they result in high compressive stresses within the pipeline.

CONCLUSIONS

Seismic design of buried pipelines is a topic of significant importance for safeguarding pipeline structural integrity of water pipelines. However, current pipeline design standards do not contain relevant provisions. The ALA (2005) guidelines, together with the Indian NICEE recommendations (2007), constitute the only complete documents on this subject, and can be used for design purposes. In the present paper, the main issues related to the mechanical behavior and strength of buried thin-walled welded steel pipelines are outlined. Special emphasis is given on pipeline resistance and the relevant failure modes. Finally, additional research is needed to determine the strength and deformation capacity of welded-slip lap joints under axial and bending loading.

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