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EXperimental & Computational Hybrid Assessment of Natural Gas Pipelines Exposed to Seismic Risk

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Seismic resilience of buried steel natural gas pipelines

Abstract. Experience has shown that earthquake damage inflicted to lifelines as pivotal as buried natural gas networks can cause long service disruptions, leading to unpredictably high socioeconomic losses in unprepared communities. Driven by this, we seek to critically revisit recent research developments in the involved field of seismic analysis, risk assessment and design of buried steel natural gas pipelines, with a view to ultimately highlighting the utility of the fast-evolving generalized concept of seismic resilience and the related progress achieved. For this purpose, we attempt to detect the critical challenges pertaining to the problem, not only from a research, but also from an industrial and regulatory point of view, elaborate on them and discuss them in a comprehensive manner. Review of the literature and the seismic code framework reveals that there are several unaddressed or unclarified issues regarding seismic analysis and risk assessment of buried gas pipelines. Furthermore, seismic resilience of buried natural gas networks still lacks thorough investigation and is completely absent from standards of practice. The future-proof goal is to move towards resilience-based code-prescribed design of buried natural gas networks.

1 Introduction

Natural gas is nowadays a cornerstone in supplying energy for industry and households, maintaining an important share in the global energy market. A steadily growing dependence of the global economy on natural gas as an energy source is reflected in the figures; one quarter of the total energy demand in the US and the European Union is currently satisfied by natural gas delivery [1,2], while it is projected that by 2040 nearly one quarter of the global electricity will be generated by natural gas [3]. Extensive onshore buried steel pipeline networks is the method of choice for natural gas distribution from source to end-users. However, of the heaviest reliers on natural gas are earthquake-prone regions, such as California, south-eastern Europe and Japan. Experience from past earthquakes has repeatedly demonstrated that buried pipelines are vulnerable to seismic effects, divided into four groups ('geohazards') based on the damage source: transient ground deformation (TGD) due to wave passage, active fault movement, landslide and liquefaction-induced settlement or lateral spread (**Figure 1**). Most of the damage reported to date has been attributed to the latter three permanent ground failures [4,5] (collectively termed PGD), but there is also strong evidence that wave propagation has contributed to substantial pipe damage [6,7].

From a system-wide perspective, the impact of a seismic shock on the network level of a natural gas pipeline system can be highly adverse and dispersed. A potential long-lasting flow disruption due to earthquake damage can have excessive direct and indirect socioeconomic consequences not only locally, but also internationally, given the spatial dimension of natural gas networks; content leakage would additionally pose an environmental threat. It becomes then evident that underground natural gas networks traversing seismically active areas are exposed to seismic risk and, therefore, efforts should be placed on securing their long-term integrity and operability with the minimum cost to society and economy. This very objective has given rise to the development of the concept of *resilience* in recent years. A number of definitions available in the literature allow a broad perception of resilience: "the capacity to cope with unanticipated dangers after they have become manifest, learning to bounce back" or "the ability to recover from some shock, insult or disturbance". Improvement of resilience is gradually being adopted as a desired target by authorities and influential movements, such as the '100 Resilient Cities' [8], within policy-making for natural disaster mitigation in urban environments.

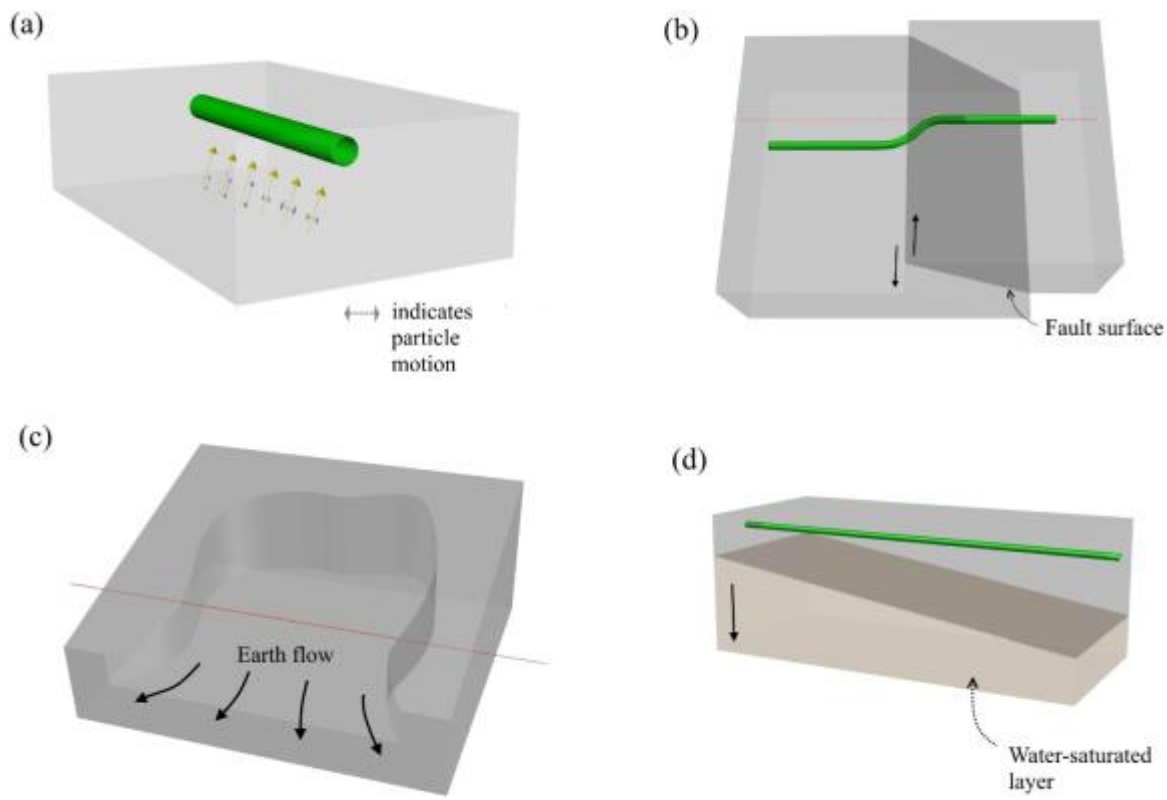

Figure 1. Illustration of the major earthquake-related geohazards threatening the structural integrity of buried pipelines; (a) seismic wave propagation; (b) strike-slip fault movement; (c) landslide in the form of earth flow; (d) liquefaction-induced settlement

The objective of the present study is to identify the primary challenges involved in the complex process of seismic risk assessment of buried steel natural gas pipeline networks from a research, industrial and legislative standpoint and review the latest progress related to them, setting seismic resilience as the utmost goal of this framework. The novelty herein lies exactly in the fact that we attempt to approach the most critical issues of seismic safety of buried natural gas pipelines through the modern prism of resilience. Previous similar efforts on this subject (e.g. [9–12]) focused exclusively on reviewing specific aspects independently of one another, lacking an holistic view of this multi-component problem.

The structure of the study is straightforward. First, six interlinked aspects of seismic analysis and risk assessment of buried steel natural gas pipelines are identified and reviewed in detail one by one, starting with component features and ending with network features. These are (1) soil-pipe interaction, (2) spatial variability of seismic ground motion along the pipeline axis, (3)

verification of dominant failure mechanisms, (4) seismic fragility expressions, (5) structural health monitoring and (6) seismic resilience. Second, existing seismic code provisions for pipeline design are assessed to conclude to what extent they meet the latest requirements proposed by research. Finally, unaddressed issues are pinpointed and discussed, and suggestions are made for future research and refinement of existing codes.

2 Dynamic soil-pipe interaction

The crucial factor that differentiates the behavior of buried structures like pipelines from that of aboveground structures is the fact that they are restrained by the surrounding soil, therefore their seismic response is largely dependent on the dynamic interaction with it. In contrast to the well-observed dynamic behaviour of aboveground structures during strong ground motion, the prevailing view about subsurface pipelines is that they are minimally affected by the earthquake inertia forces, for these are resisted to a great extent by the surrounding soil mass. This statement, recognized by researchers and reflected in design codes [13–16], implies that inertial soil-pipe interaction effects, as they occur in aboveground structures, are practically insignificant. When an earthquake strikes and travelling seismic waves arrive at a point along a pipeline, it is the relative movement between the affected pipe segment and the soil that primarily contributes to the development of stress in the pipe and incurs structural damage. For this reason, force-based analysis methods are not recommended for the design of buried pipelines, rather a need to ensure code-prescribed ductility levels arises in this instance.

2.1 Models ignoring soil-pipe interaction

A quite common, yet seemingly sound assumption adopted both in design practice and research (e.g. [17–19]) is that the soil around the pipe possesses considerably greater stiffness than the pipe itself, hence the latter is actually forced to perfectly conform to soil movement. From this assumption, it follows that pipe strains match soil strains. This approach is apparently conservative, because it permits pipe designs for higher strains than it would be the case if the pipe could resist soil distortion [20]. Again, it is important to understand that enhancing pipe strength is not effective, and the principal design criterion should be to maximize the ductility of the pipe. This is underpinned by the fact that pipelines made of cast iron (a non-ductile material) have suffered extended damage compared to steel pipelines in past earthquakes.

In this respect, fundamental was the early approach proposed by Newmark [17]. Based on the simplification that ground shaking is triggered by a single shear wave train and the theory of wave propagation in an infinite, homogeneous, isotropic, elastic medium, he developed the following analytical strain expression, also recommended by Eurocode 8:

$$\frac{\partial u}{\partial x} = -\frac{1}{V_{app}} \frac{\partial u}{\partial t} \tag{1}$$

where $\partial u / \partial x$ represents the free-field strain in the direction of propagation and $\partial u / \partial t$ is the particle velocity. Eq. (1) can be manipulated to determine the strains in a buried pipe struck by P- or S-waves under the assumption of soil-pipe interaction absence. Kuesel [18] implemented this approach for the earthquake-resistant design of the San Francisco Trans-Bay Tube. However, Newmark's simplified approach yields credible results only for highly flexible pipes. Large-diameter pipelines, such as natural gas transmission pipelines, possess stiffness that prevents them from conforming to soil motion; hence applying Newmark's approach in this case would lead to overdesign.

2.2 Models with account for soil-pipe interaction

When pipeline stiffness is appreciable with respect to that of the soil, as in soft soils or large-diameter and thick-walled pipes, pipeline movement deviates from ground movement; soil-pipe interaction effects are likely to play a critical role in the response of the pipeline in this instance. Several mathematical models for considering the interaction in the soil-pipe system have been proposed in the literature, ranging from simple to more advanced ones.

The most frequently encountered approach, and the simplest one, involves application of the beam-on-nonlinear-Winkler-foundation (BNWF) model. In it, the pipeline is represented by elastic beam elements, while discrete equivalent translational springs, characterized by appropriate stiffness, are assigned at points along its axis in each principal direction to model the behavior at the soil-pipe interface. In a one-dimensional treatment of the complete dynamic problem, the governing equation of motion of a pipeline excited by a ground displacement time history $w_g(t)$ in the transverse horizontal direction is

$$m\frac{\partial^2 w}{\partial t^2}+c_h\frac{\partial w_r}{\partial t}+k_h w_r+EI\frac{\partial^4 w}{\partial x^4}=0 \tag{2}$$

where w represents the time-dependent pipe transverse displacement, $w_r = w - w_g$ is the relative transverse displacement between the pipe and the ground, m is the distributed mass along the pipeline, EI is the flexural rigidity of the cross-section, c_h and k_h are the dashpot and spring constants per unit length of the pipeline in the transverse horizontal direction. If the dynamic effects are ignored, quasi-static response governs and Eq. (2) becomes

$$EI\frac{\partial^4 w}{\partial x^4}=k_h\left(w_g-w\right) \tag{3}$$

Similarly, the quasi-static response in the axial direction is described by the following equation:

$$EA\frac{\partial^2 u}{\partial x^2}=k_a\left(u-u_g\right) \tag{4}$$

where $u-u_g$ is the relative axial displacement between the pipeline and the ground, EA is the axial rigidity of the pipe cross-section and k_a is the spring constant in the axial direction per unit length of the pipeline.

In an early study, St. John and Zahrah [20] derived a reduction factor to estimate the internal forces of an interacting soil-pipe system from that of a corresponding interaction-free system, making simplified assumptions regarding the nature of the oncoming seismic waves. The interpretation of this reduction factor is that accounting for the soil-pipe interaction effects has a favourable effect on the pipe forces. That statement is further supported by another interesting study conducted by Hindy and Novak [21]. In this study, a lumped mass beam-model for the pipe was adopted and analyzed considering dynamic soil-pipe interaction, similarly to the continuous problem described in Eq. (2). Two different soil configurations were examined; in the case of a homogeneous medium, it was found that soil-pipe interaction leads to decreased pipe stresses as compared to the ones obtained neglecting it, while in the case of a soil consisting of two different layers separated by a vertical plane, stress concentration was located close to the vertical boundary and the pertinent peak values were even higher than the ones predicted without

soil-pipe interaction. In a similar study [22], Parmelle and Ludtke conclude that the effect of soil-pipe interaction is negligible.

The fundamental challenge in representing the soil-pipe interaction with equivalent soil springs is to determine their nonlinear behaviour in a reliable way. This has been a subject of continuous research over the years and substantial progress has been achieved, providing mainly elastoplastic idealizations of the true nonlinear soil response. **Table 1** summarizes the soil-pipe interaction models presented next.

One of the first known such tests was conducted by Audibert and Nyman [23], who studied the lateral (horizontal) response of steel pipelines buried in sand under a wide range of burial depth to pipe diameter ratios and developed a rectangular hyberbola for modelling the soil resistance as a function of the relative lateral movement. Their proposed ultimate soil resistance against lateral pipe motion is given by:

$$F_{U,lateral} = \gamma' D H N_q \tag{5}$$

where γ' is the effective unit weight of the soil, D is the outside pipe diameter, H is the depth to the pipe centreline and N_q is the bearing capacity factor, estimated from appropriate charts.

Later, Nyman [24] investigated the restraints induced in cohesionless soil due to oblique vertical-horizontal pipe motion. Extending the solution of Meyerhof [R] for inclined strip anchor resistance, he proposed an expression for the ultimate soil restraint against the oblique pipe motion as the product of the ultimate soil restraint against vertical pipe motion $F_{\mathrm{U,vertical}}$ and an inclination factor $\boldsymbol{R}_i$:

$$F_{U,oblique} = R_i F_{U,vertical} \tag{6}$$

with $$R_i = 1 + \frac{0.25a}{90^\circ - 0.75a}\left(\frac{F_{U,lateral}}{F_{U,vertical}} - 1\right) \tag{7}$$

where a is defined as the inclination angle in degrees between the oblique and the vertical soil restraint and $F_{\mathrm{U,lateral}}$ can be evaluated from (5) or other sources. To completely describe the

nonlinear force-displacement relationship, Nyman recommends the following values for the yield displacement of the soil that is required to mobilize the oblique ultimate soil restraint:

$$\delta_{y,oblique} = \begin{cases} 0.015H & \textit{for dense geomaterials} \\ 0.025H & \textit{for loose geomaterials} \end{cases} \tag{8}$$

To validate the available analytical models against experimental data, Trautmann and O' Rourke [25] performed a series of multi-parametric lateral loading tests to assess the response of subsurface, typical-sized pipelines to lateral soil motion. A hyperbolic function was derived to represent the average lateral force-displacement curve of the obtained test data, expressed in dimensionless form as:

$$F/F_U = \frac{\delta/\delta_y}{0.17 + 0.83\delta/\delta_y} \tag{9}$$

where $F_U = \gamma HDLN_h$ is the ultimate soil force, with L and N_h standing for the length and the horizontal force factor, respectively. Appropriate values for the latter parameter may be sought in relevant charts as a function of depth-to-diameter ratio and friction angle. Test results also indicated a strong variation of the yield displacement δ_y of the soil with the soil density, ranging from $0.13H$ for loose soil to $0.08H$ for medium soil to $0.03H$ for dense soil.

In order to characterize the transverse horizontal and axial soil movement described in Eqs. (3) and (4), St. John and Zahrah [20] used a foundation modulus obtained by manipulating the solution to the Kelvin's problem of a point static load applied within an infinite, homogeneous, elastic, isotropic medium. The result was expressed as:

$$k_a = \frac{16\pi}{\ell} \frac{(1-\nu)}{(3-4\nu)} GD \tag{10}$$

where ν, G are the Poisson's ratio and shear modulus of the medium and D the outer pipe diameter. In the same manner, but utilizing the solution to the Flamant's problem, they arrived at an estimate for the foundation modulus that governs the pipe response to transverse vertical soil motion:

$$k_v = \frac{2\pi G}{(1-v)} \frac{D}{\ell} \tag{11}$$

Concerned with the evaluation of axial soil springs, El Hmadi and O' Rourke [26] attempted to verify the theoretical and empirical predictions for the axial spring stiffness available at that time, taking advantage of the experimental data provided by a previous full-scale field test [27]. After performing a back-calculation on the governing displacement functions and also considering the strain-dependent nature of the soil shear modulus, they ended up with an upper and lower bound value for the axial spring constant k_a as a function of the soil shear modulus G :

$$1.57G \le k_a \le 1.70G \tag{12}$$

This range of values apparently lies within and consequently partly confirms the wider range provided by the then existing literature $G \le k_a \le 3G$. Another important finding of this study is that the inertial axial force induced in the pipeline during the test was over two orders of magnitude lower than the soil restraint developed, thus verifying the statement that pipeline inertia is insignificant.

O' Rourke and El Hmadi [15] established among others a relationship for the maximum frictional resistance per unit length that develops at the soil-pipe interface under relative axial motion between the soil and a pipeline with sand backfilling, considering that by definition this is given by the product of the applied vertical force and the coefficient of friction. It may be estimated as follows:

$$F_{U,axial} = \mu \gamma' H \left(\frac{1+k_0}{2} \right) \pi D \tag{13}$$

where μ represents the coefficient of friction, k_0 is the coefficient of lateral earth pressure and πD the circumference of the pipe.

In a later experimental effort, Hsu *et al.* [28] dealt with the response of pipes buried in loose sand and subjected to oblique-horizontal increasing displacement. In specific, a large-scale test was carried out involving various pipe specimens and depth configuration, wherein the pipe was

successively placed at horizontal orientations in different, gradually increasing inclination angles with respect to the direction of movement. The goal was to evaluate the longitudinal and transverse horizontal (lateral) soil restraint components for each test setup of the oblique pipe. Their results indicated that the axial-oblique restraint can be determined simply by multiplying the axial force of the corresponding purely axial pipe ($a = 0°$) with the cosine of inclination angle, while to obtain the lateral-oblique restraint, a multiplication between the lateral force of the associated purely lateral pipe and the sine of the inclination angle is sufficient:

$$F_{axial-oblique} = F_{axial} \cos a \tag{14}$$

$$F_{lateral-oblique} = F_{lateral} \sin a \tag{15}$$

where a is the inclination angle between the orientation of the pipeline and the direction of

Table 1. Soil-pipe interaction models proposed in the literature, including ultimate soil force and equivalent elastic spring stiffness relationships (all parameters and variables involved are explained in the body).

Reference	Relationship	Remarks
Audibert and Nyman (1977)	$F_{U,lateral} = \gamma' D H N_q$	For lateral loading of steel pipe in sand
Nyman (1984)	$F_{U,oblique} = R_i F_{U,vertical}$	For oblique loading in cohesionless soil
Trautmann and O' Rourke (1985)	$F/F_U = \dfrac{\delta/\delta_y}{0.17 + 0.83\delta/\delta_y}$	For lateral loading; δ_y ranges from to $0.03H$ to $0.13H$ depending on soil density
St. John and Zahrah (1987)	$k_a = \dfrac{16\pi}{\ell} \dfrac{(1-\nu)}{(3-4\nu)} GD$	For axial loading; elastic soil response
El Hmadi and O' Rourke (1988)	$1.57G \le k_a \le 1.70G$	For axial loading
O' Rourke and El Hmadi (1988)	$F_{U,axial} = \mu\gamma' H \left(\dfrac{1+k_0}{2}\right)\pi D$	For axial loading; sand backfill
Hsu *et al.* (2001)	$\begin{cases} F_{axial-oblique} = F_{axial} \cos a \\ F_{lateral-oblique} = F_{lateral} \sin a \end{cases}$	For oblique loading in loose sand

movement.

The American Lifeline Alliance presented a report [29] that contains mathematical expressions for describing the behaviour of nonlinear soil springs in each of the four principal directions of pipe motion, i.e. axial, lateral, vertical uplift and vertical bearing. In all cases, the nonlinearity of the soil is idealized by an elastoplastic bilinear curve, hence only one point is actually needed to define each curve. These models provide a way to estimate both the maximum soil restraints and the corresponding relative displacements. The relationships (**Table 2**), extensively used in design practice, were derived assuming uniform soil conditions and are mainly based on Refs. [30,31]. Nourzadeh and Takada [7] use these relationships to generate soil spring models in their numerical parametric investigation of the response of buried steel gas pipelines to seismic wave propagation. Beam elements are used to model the pipeline and three-component displacement time histories are used as seismic input. Analyses show that pipelines experienced at least local buckling under PGAs greater than 0.6g; however, performance criteria are too loosely defined to allow safe judgment.

Further, a number of recent studies have explored through analytical or numerical approaches the response of buried steel pipelines to various types of tectonic fault movements, considering the soil-pipe interaction. Karamitros *et al.* [32], extending the work by Kennedy et al. [R],

Table 2. Ultimate soil force and relative displacement relationships for soil-pipe relative motion proposed by the ALA [29].

Spring direction	Ultimate soil restraint	Ultimate relative displacement
Axial	$\pi Dac + \mu\gamma' H\left(\frac{1+k_0}{2}\right)\pi D$	$3 \div 10\, mm$ *depending on soil stiffness*
Lateral	$cDN_{ch} + \gamma' DHN_{qh}$	$0.04(H + D/2) \le 0.10D \sim 0.15D$
Vertical uplift	$cDN_{cv} + \gamma' DHN_{qv}$	$\begin{cases} 0.01H \sim 0.02H < 0.1D & \text{dense to loose sands} \\ 0.1H \sim 0.2H < 0.2D & \text{stiff to soft clays} \end{cases}$
Vertical bearing	$cDN_c + \gamma' DHN_{qb} + \gamma \frac{D^2}{2} N_\gamma$	$\begin{cases} 0.1D & \text{for granular soils} \\ 0.2D & \text{for cohesive soils} \end{cases}$

a : adhesion factor; c : backfill soil cohesion; N_{ch}, N_{qh}, N_{cv}, N_{qv}, N_{qb}, N_c, N_γ : bearing capacity factors in the horizontal, vertical uplift and vertical bearing direction (subscripts c denotes clay, q denotes sand)

developed an analytical design methodology to estimate pipeline axial and bending strains generated by strike-slip fault movement. In a series of studies [33,34], Vazouras *et al.* used rigorous shell and solid finite elements for the soil-pipe model to study numerically the nonlinear behaviour of buried steel pipelines crossing obliquely active strike-slip faults, also considering the influence of pipe continuity by deriving special spring relationships for the model ends. Sarvanis *et al.* [35] proceeded to build advanced finite element models for the soil-pipe interaction problem in the axial and transverse direction by calibrating the involved parameters through full-scale tests. Analysis of the calibrated numerical model under fault movement was performed and results were compared to full-scale fault experiments, showing good agreement in terms of axial strains. From a different perspective, Karamanos [36] points out that pipe elbows exhibit more flexible behaviour compared to straight pipe parts and are more prone to section ovalization due to bending and fatigue damage under cyclic loading. These facts render pipe elbows the most critical components in a pipeline.

Remarks

A common deficiency in the majority of the cited studies (with few exceptions, see [37]) is that the potential role of the kinematic part of interaction in the seismic response of the pipe is not examined at all. More importantly, experimental studies dealing with the derivation of force-deformation relationships for the soil springs are usually based on static loading tests, hence they are most applicable to cases of earthquake-induced PGD. This, however, contradicts with the true, cyclic nature of seismic excitation; hysterisis characteristics of both the pipe and the soil are neglected. In view of this, emphasis should be placed on the development of reliable cyclic force-displacement curves that describe the dynamic interaction between pipe and soil under seismic shaking. Another assumption often used is the homogeneity of the medium along the pipeline route, which apparently does not hold true considering that pipelines are geographically distributed systems. Different lateral soil conditions might significantly affect the stress distribution in the pipeline, as already indicated in some studies (e.g. [21,38,39]). This issue requires further investigation in the framework of full dynamic soil-pipe interaction analysis under the assumption of horizontally varying soil composition.

3 Predicting spatially variable transient ground motions along the pipeline

Spatial variability in earthquake ground motion can be interpreted as the differences expected in frequency content, amplitude and phase angle of seismic signals captured at distant stations on a local scale; this observation was consolidated over three decades ago, when researchers [40][41] started analyzing the ample accelerogram data obtained from densely installed strong motion recording arrays, in particular the SMART-1 array in Taiwan. Spatial variability is a physical phenomenon of stochastic nature, in the sense that its occurrence can only be predicted with a degree of uncertainty due to the complex, multi-parametric underlying mechanisms that contribute to its generation.

These variations in the seismic ground motion are principally attributed to three factors [42]: (a) the transmission of the waves at finite velocity (also known as the *wave passage effect*), which intuitively results in different arrival times at different recording stations, (b) the gradual reduction in the coherency of the waves as a result of successive scattering, such as reflections and refractions, that occurs along their path through the non-homogenous earth strata (*ray-path effect*) and due to the varying superposition of waves originating from different points of an extended seismic source (*extended source effect*), collectively known as the 'incoherence effect', (c) the different local soil conditions at remote stations that primarily affect the amplitude and frequency content of the incoming waves (*local site effect*). Additional causes of the phenomenon have also been recognized: the attenuation of seismic waves along their path, resulting from the gradual dissipation of wave energy into the soil medium, and the relative flexibility of the soil-foundation system that may 'filter' certain frequencies of the incoming wave field [43]. However, the influence of the latter two sources is usually ignored in modeling spatial variation of seismic ground motion as it is generally regarded insignificant.

Spatial variability in ground motion has been rigorously investigated by modeling the earthquake ground acceleration as a random signal of time. Descriptors of the probabilistic properties of the ground motions have been established and used to reflect the sources of spatial variability [44]. Random vibration analysis or deterministic time-history analysis using simulated spatially variable ground motions as input are employed to assess the effect of the phenomenon on the response of various structures.

Relatively limited research has been reported to date on the effect of spatial variation in ground motion on the response of pipelines, over-ground or underground. Zerva *et al.* [45] examined in the stochastic domain the axial and transverse response of segmented and continuous pipelines of various lengths to differential ground motion; in this framework, they performed random vibration analysis of analytical pipeline models using as input the stochastic properties of ground motions recorded at the SMART-1 array. Results for partially coherent motions were compared to the ones corresponding to perfectly coherent motions. For the case of segmented pipelines, it was shown that both axial and lateral responses are comparable and an analysis using homogeneous earthquake input would result in a stress-free pipe state. For the case of continuous pipelines, a close match was found between displacements obtained for fully and partially correlated motions, a circumstantial finding attributed to the fact that the same rigid body mode was excited in both cases; however, partially correlated motions gave higher stress values. Further, it was observed that axial stresses are becoming dominant as the slenderness of the continuous pipeline increases, while bending stresses become sizable when the pipe diameter is large. Another probabilistic study with similar objectives was conducted by Zerva *et al.* [46] yielding pipeline response statistics that confirmed the conclusions noted in their previous report [45]. In a later effort, Zerva [47] investigated the effects of directionally and spatially correlated seismic ground motions on the response of continuous, large-diameter pipelines through random

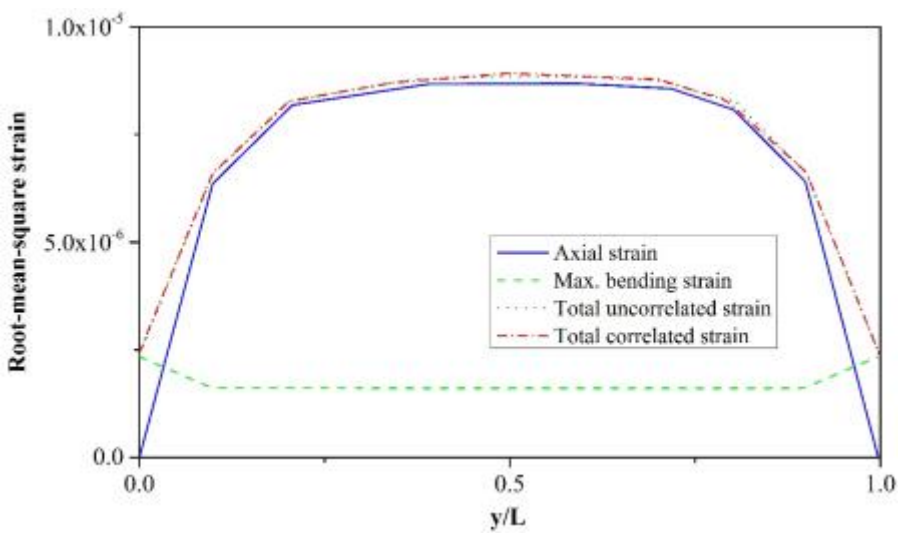

Figure 2. Distribution of root-mean-square strains along the pipeline axis for pipe orientation that coincides with the epicentral direction of the input motion, as calculated by Zerva (1993) (adapted from [47]).

vibration analyses using similar analytical formulations as before and seismic input represented by stochastic characteristics recorded at the SMART-1 array. The scrutiny revealed that considering the correlation between the two horizontal seismic motion components provides negligible discrepancies and that axial strains are the principal source of pipeline deformation over bending strains (**Figure 2**); it was also shown that the selection of the incoherence parameter on the pipeline response is critical (**Figure 3**).

Zerva [48] dealt with the effect of differential ground motions on the response of various lifeline structures, including underground pipelines. By approximately estimating the seismic axial strains along a buried pipeline model using two coherency decay models ([49], [50]), she noted that these obtain their maximum values when the motions are totally incoherent, i.e. the differential displacements at the input stations are maximum. Specifically for the second coherency model, she observed an increasing trend in the seismic strains with increasing value of the decay parameter a (denoting increasing incoherence). To further support the significance of the incoherence effect, Zerva performed a comparative study to determine its relative influence with respect to the wave passage effect. It was found that in the case of high apparent wave propagation velocity, seismic strains along the pipeline are primarily controlled by the degree of incoherence of the motions. On the other hand, for relatively lower values of apparent propagation velocity, seismic strains are proportional to the reciprocal of this velocity. The study

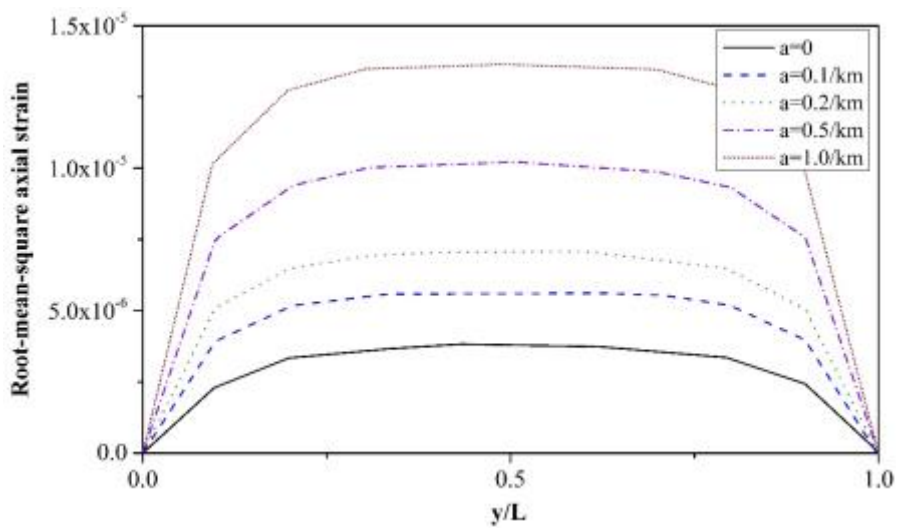

Figure 3. Distribution of root-mean-square axial strains along the pipeline for different values of the incoherence parameter, as calculated by Zerva (1993) (adapted from [47]).

by Lee *et al*. [51] uses multiple seismic excitation along a BNWF model of a pipeline in a 3D nonlinear time-history analysis, showing that the pipeline presents varying distribution of the axial relative displacement along its length, with peaks appearing in the region of differing imposed excitations. As regards the transverse response, calculated pipeline demand for a specific input ground motion reached half the respective capacity. It is also underlined that transverse response under multiple excitation is affected by soil conditions.

Notwithstanding these pioneer previous studies, the conclusions drawn cannot be generalized to describe the seismic response of buried pipelines to spatially variable ground motion, mainly for three reasons. First, the results are plausibly specific to the recorded ground motion stochastic characteristics selected for input. Second, the response of the pipeline depends highly on the coherency model used in the analysis, which in turn has dependence on various incoherence parameters. Third, the investigation so far is constrained in theoretical boundaries; further evidence through laboratory work is deemed necessary in order to assess the degree to which buried pipeline networks are vulnerable to differential seismic ground motions, especially considering potential heterogeneities in soil conditions (local site effects), towards the verification of the existing theoretical and numerical findings.

4 Dominant failure modes and supporting evidence from past earthquakes

In the course of earthquake-resistant design of underground pipeline networks, the principal mechanisms that lead to failure due to seismic excitation have to be identified in order for appropriate performance criteria to be established. Extensive previous research efforts and field surveys have been successful in identifying the most frequently occurring failure modes, classified into two groups: those observed in continuous and those observed in segmented pipelines. The first group includes pipelines assembled with welded connections equally strong or stronger than the pipe barrels, while the second group includes pipelines in which mechanical joints are the weak link of the chain due to their lower strength. Herein, discussion is focused on continuous pipelines. Assuming a flawless welding process and corrosion-free conditions, one can distinguish five failure mechanisms for continuous steel-welded pipelines triggered by ground shaking or PGD: pure tensile rupture, local bucking, upheaval buckling, flexural failure and section distortion [52,53] (see **Figure 4**).

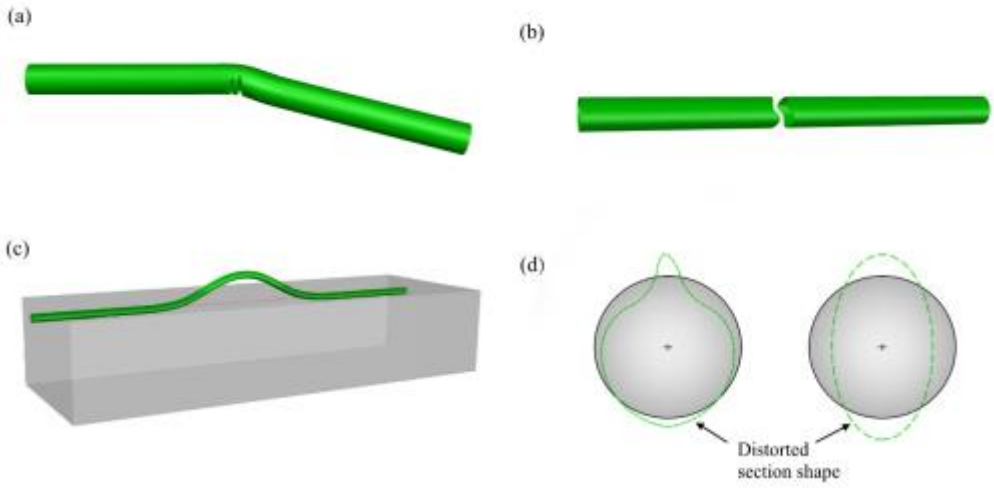

Figure 4. Frequently observed failure mechanisms in buried continuous steel pipelines: (a) local buckling; (b) tensile fracture; (c) upheaval buckling; (d) cross-section distortion

Tensile fracture

When excessive plastic longitudinal strains accumulate in pipe walls, rupture is expected to occur. This type of failure is rarely observed in arc-welded steel pipelines with butt connections, as these exhibit a strongly ductile behaviour. On the contrary, steel pipelines assembled with gas-welded slip joints are more vulnerable to this failure mechanism, since they are incapable of withstanding that substantial yielding before tensile rupture. This exact finding is concluded in [54], based on relevant evidence from the 1994 Northridge event.

Although the fracture strain of X-grade pipe steel may well reach 6% [55], usually a more conservative value of 3% [16,56] or 4% [52] is adopted in engineering practice. In general, experience from previous earthquake events has shown that most steel pipelines exposed to tensile loading performed more than sufficiently, since modern manufacturing techniques are able to satisfy ductility requirements.

Local buckling

Local buckling (or shell wrinkling) is a failure state associated with structural instability issues appearing under pipe compression. In essence, it involves localized distortion of the pipe wall, which in turn can lead to further curvature amplification in that region and tearing. Local buckling is a common failure mode in steel pipelines, as indicated by observations of pipeline performance in past earthquakes [52]. Specifically, local buckling caused by wave propagation

affected a water pipe during the 1985 Michoacan event in Mexico City, whilst liquid fuel, water and gas pipelines were found to suffer such damage as a result of PGD in the 1991 Costa Rica and 1994 Northridge earthquakes. More than that, local buckling due to PGD was evident in pipelines crossing faults, both normal and reverse, in the 1971 San Fernando event. Experience so far shows that local buckling distortions tend to accumulate at geometry transition regions of the pipeline, such as bends and elbows.

Hall and Newmark [57], based on previous experimental results, recommended a failure criterion for pipe local buckling by determining a range in which lies the critical compressive strain corresponding to the onset of shell wrinkling. Later, this criterion was adopted as a design provision by ASCE [58]. The range is expressed as:

$$0.30(t/D) \le \varepsilon_{cr} \le 0.40(t/D) \tag{16}$$

where ε_{cr} is the critical compressive strain marking the start of buckling, t and D are the wall thickness and the outer diameter of the pipe, respectively. O'Rourke and Liu [52] note that the above criterion finds better applicability to thin-walled pipes, while it is rather conservative for thick-walled ones. Vazouras *et al.* [33] also establish a 'no-buckling' requirement for buried pipelines deformed by strike-slip fault movement normal to their axis. After deriving a simplified expression for the peak compressive pipe strain based on the assumption of a fixed deformed shape and adopting a general form for the critical buckling strain as a function of the thickness-to-diameter ratio, they arrived at the following limit condition:

$$D/t \le 0.4a(L/D)^2 \tag{17}$$

where L is the length of the deformed segment of the pipeline and a is a parameter depending on the pipeline material and initial imperfections.

Upheaval buckling

Steel pipelines subject to compressive ground forces are also likely to suffer from upheaval (sometimes referred to as beam) buckling, a failure mode that resembles the well-known Euler buckling of a column. In this failure mechanism, compressive strains are not confined in short zones of the pipe walls, as in local buckling, rather they are distributed over greater lengths, at a global level. For this reason, the likelihood of pipe breakage is generally lower than in the case

of local buckling, therefore upheaval buckling is a less catastrophic type of failure [52]. That said, upheaval buckling is better characterized as a serviceability peril and not as a classic material-related failure, since the pipeline can continue transmitting its contents along its extent. On this basis, a criterion describing the limit state of a pipeline just before upheaval buckling occurs is dependent on several parameters, such as the flexural rigidity of the pipe section, potential structural imperfections and the burial depth of the pipeline, and consequently is difficult to develop.

Meyersohn and O'Rourke [59] noticed that pipelines covered by backfill soil with limited uplift resistance are more likely to fail by means of upheaval buckling. They pointed out that there is a proportional relationship between buckling load and trench depth and calculated a critical value for the latter, which in effect determines the precedence of occurrence of the two modes of buckling; that is, if a pipeline has a larger burial depth than the critical cover depth, then local buckling will occur before upheaval buckling and vice versa. Further on this, it was noted that a minimum cover depth of 0.5 to 1.0 m is sufficient to ensure that the pipeline will not experience upheaval buckling.

Observations from previous earthquakes reveal that upheaval buckling has indeed affected underground pipelines in some cases. In 1959, oil pipelines covered with a shallow trench with a depth ranging between 0.15 and 0.30 m and traversing the Buena Vista reverse fault lifted out of the ground because of high compressional stresses. In another interesting occasion related to the 1979 Imperial Valley seismic event, no evidence of upheaval buckling in two pipelines crossing the fault was available until local inspections by means of cover removal forced the pipelines to buckle upwards [60]. This is also an indication that upheaval buckling may not always interrupt the functionality of the pipeline.

Flexural failure

Failure due to excessive bending of the pipe section is quite rare in steel pipelines because of the high ductility of steel. To this conclusion points among others evidence from the 1971 San Fernando earthquake event, where a number of buried gas and liquid fuel pipelines were found to have endured approximately 2.5 m of transverse soil displacement [61].

Section distortion

Another possible failure associated with large radial deformations is the cross-sectional distortion or 'ovalization', as is the term most frequently used. Severe bending may force the pipe circular cross-section to flatten into an oval-like shape, which can pose a serviceability threat to the pipeline carrying capacity. The limit state for this failure mode has been codified by Gresnigt [62] through a critical change in the pipe diameter ΔD_{cr} as:

$$\Delta D_{cr} = 0.15D \tag{18}$$

It is important to emphasize that a different approach to the establishment of failure criteria is expected to be followed for continuous pipelines with slip, riveted or gas-welded joints. As opposed to pipelines assembled with butt joints, for which failure criteria are mostly functions of pipe performance indicators, in this case failure criteria have to be formulated on the basis of joint characteristics, because this type of joints are generally weaker than the main pipe body. A number of studies involved the estimation of the strength of slip joints with inner [63,64] and outer weld [65] in terms of joint efficiency, namely the ratio of joint to pipe strength. Joint efficiency values lower than 0.40 were obtained in all cases. Damage evidence at welded joints is available from the 1971 San Fernando earthquake, where most of the failures were observed at the welds of gas-welded joints.

Recent damage observations and remarks

Damage in buried pipelines caused by recent major earthquakes is a subject of current scrutiny. Esposito *et al.* [66] recorded a significant level of damage in gas-welded steel joints in the local underground gas distribution network after the 2009 L'Aquila event. This damage is described as breaks or leaks, but no further details as to the exact failure modes are provided. Koike *et al.* [67], in estimating the seismic performance of the gas pipeline network following the devastating 2011 Tohoku event in Japan, note that high-pressure transmission pipelines survived successfully the impact of the earthquake with only minor damage, even in mountain settings where landslides occurred. More recently, Edkins *et al.* [68] identified the characteristic failure mechanisms affecting buried pipelines of different materials, based on interpretation of photographic material obtained after the 2010/2011 Canterbury earthquakes. They conclude that different failure modes may occur depending on the material, the soil conditions, direction of excitation and pipeline size. The samples examined do not include any steel pipelines, though.

Focusing attention on steel pipelines, which typically make up for the largest part of gas transmission networks, it becomes clear that the existing failure criteria lack robust scientific basis. Physical testing of specimens under controlled laboratory conditions is necessary in order to determine limit state parameters governing different failure modes and also clarify the influence of factors such as soil conditions and pipeline size. Furthermore, when performing numerical investigations, emphasis should be placed on the selection of the finite element model; failure states like local buckling and section deformation cannot be predicted by simple beam models, as this requires the adoption of more sophisticated cylindrical shell models.

5 Fragility expressions for buried pipelines

In the last decades, a gradual transition is seen in the interest of the structural engineering community from conventional deterministic analysis procedures to probabilistic risk assessment concepts, as the understanding of how various uncertainty sources, both aleatory and epistemic, may affect the basic variables governing the response of structures to natural hazards is becoming more profound and the available computational capabilities are rapidly evolving. Particularly in earthquake engineering problems, wherein uncertainties due to the nature of the hazard are amplified, structural reliability tools have drawn significant research attention lately in an attempt to quantify these uncertainties, explore their potential propagation throughout the model and evaluate the risk level the structure is exposed to. When it comes to the seismic safety of infrastructure of paramount civil importance, such as utility systems, probabilistic approaches are deemed more than necessary to secure minimum functionality disruption and overall longevity under different excitation levels.

In a broad context, a fragility curve expresses the conditional probability that a structural system or individual component of the system reaches or exceeds a certain limit damage state for a given load intensity. This probability measure is commonly referred to as the probability of failure, where the term ‘failure’ does not necessarily imply catastrophic damage, rather refers to different predefined levels of so-called unsatisfactory performance. In the sphere of earthquake engineering, fragility curves are used to investigate the probability that the imposed seismic demand D is equal to or greater than the capacity C corresponding to a specified damage state of the structure, given a ground motion intensity measure (IM hereafter) magnitude, according to the following probability statement:

$$Fragility = P\left[D \geq C \mid IM\right] \tag{19}$$

In the context of damage analysis of buried pipelines, probabilistic expressions known as seismic fragility relations are the typically used evaluation tool. They establish a relationship between the spatially distributed pipe damage rates and the different degrees of earthquake severity. The damage rate is usually quantified as the pipeline repair rate, i.e. the number of pipe repairs (breaks or leaks) per unit length of pipelines, although other measures have also been used. Seismic fragility relations are usually categorized according to the damage source, that is, TGD and PGD, and are written as:

$$RR = aIM^{b} \tag{20}$$

where RR is the median repair rate and a and b are parameters estimated from regression analysis of the available data pairs.

Several different ground motion IMs have been claimed in the literature to correlate well with pipeline damage, ranging from the generally adopted MMI, PGA, PGV, AI, $S_a(T_1)$ to the more pipeline-specific peak ground strain ε_g and PGV^2 / PGA. The majority of studies on seismic fragility of buried pipelines adhere to the fragility relation scheme based on collected empirical data. At least to the authors' knowledge, only few research efforts [69,70] have advanced to producing classic fragility curves by calculating probabilities of failure.

Empirical seismic fragility relations for buried pipelines

The first studies that utilized observed pipe damage from earthquakes date back to 1975, when Katayama *et al.* [71] published charts of pipe damage as a function of PGA for different soil categories, taking into account data obtained from six events. Later, Eguchi [72] generated expressions for pipe breaks in terms of the MMI scale for various pipe materials, being the first to distinguish between wave propagation and PGD hazards and providing a ranking in terms of vulnerability of different pipe materials as follows (in descending order): concrete, PVC, cast iron, ductile iron, X-grade steel. Barenberg [73] and Ballantyne *et al.* [74] first developed fragility relations considering PGV as the ground motion IM. Along the same lines, empirical PGA-based fragility expressions were produced in three subsequent studies [75–77].

A remarkable effort in the literature is that of M. J. O' Rourke and Ayala [78], who proposed a PGV-based seismic fragility relation based on damage data associated with pipelines of various materials from three earthquake events. Their function concerning damage due to wave propagation was adopted by FEMA in HAZUS methodology [79]. Further on this subject, T. D. O'Rourke *et al.* [80] performed comparative damage analyses using different IMs; their conclusion was that the highest correlation between damage and seismic severity is achieved with the use of PGV as IM. In an alternative approach, Trifunac and Todorovska [81] defined the damage rate as the amount of pipe breaks per square km of land area and used the peak soil shear strain as IM to derive fragility expressions for water pipelines based on the 1994 Northridge event. T. D. O'Rourke and Jeon [82] used cast iron pipe damage evidence from the 1994 Northridge event to develop a fragility relation for wave propagation.

In a relevant guideline document [83], the American Lifeline Alliance incorporated the most comprehensive list of seismic fragility relations for water supply pipelines, based on an extensive database of documented damage that includes 81 data points. The relations are provided in the form of backbone functions, allowing for adjustment through correction factors to account for different pipe materials, joint types and other parameters, and their validity has been confirmed in practice in recent earthquake events. It should be noted that the damage data present considerable scatter and, moreover, refer mostly to cast iron and asbestos cement pipeline. In another notable publication, M. J. O' Rourke and Deyoe [84] accomplished a twofold objective, re-examining previously used data sets related to segmented buried pipes: on the one hand to illustrate that the peak ground strain ε_g is more consistent than PGV in describing seismic damage to segmented buried pipes, on the other hand to develop improved fragility relations in terms of ε_g and also PGV-based relations considering the type of the controlling seismic wave. These relations are based on the assumption that S-waves govern for near-source sites and R-waves for far-source sites. Jeon and T. D. O'Rourke [85] performed comparisons among damage prediction equations using differently estimated PGV, concluding that the maximum recorded PGV value provides better correlation with water supply pipeline damage rates. Later, Pineda and Ordaz [86] proposed a new IM for buried pipeline fragility functions, PGV^2 / PGA, and showed that it is more closely related to damage patterns in soft soils. By assuming different

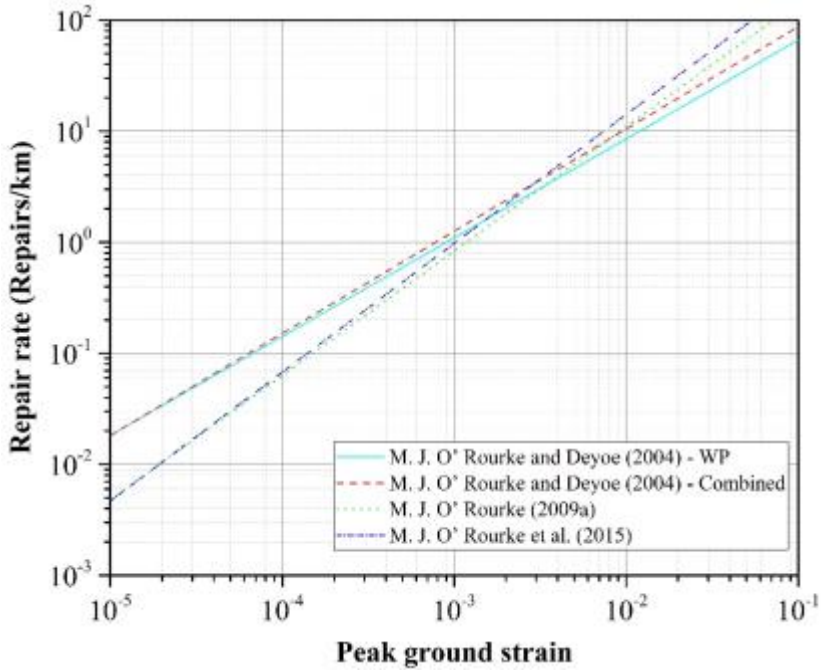

Figure 5. Comparative log-scale plot of published strain-based empirical fragility relations for buried pipelines.

effective wave velocity, M. J. O' Rourke [87] presented a revised strain-based fragility relation for segmented pipes exposed to seismic wave propagation.

Esposito *et al.* [66] presented a comprehensive study analyzing the performance of the L'Aquila medium- and low-pressure gas distribution network in the 2009 earthquake. Relying on damage reports, seismic fragility of buried steel pipes in terms of repair rates was estimated and plotted against local-scale PGV values interpolated using Shakemap tools. Then, the obtained data were validated against existing fragility relations, giving non-negligible damage underestimations by the latter. The deviations were attributed to the fact that the fragility relations used were established for arc-welded steel pipes, while the L'Aquila gas pipeline network consists of gas-welded pipes, which are more vulnerable. Noteworthy is the fact that HDPE pipes exhibited no damage at all. More recently, T. D. O'Rourke *et al.* [88] assessed the performance of underground water, wastewater and gas pipelines during the 2011 Canterbury seismic episodes. By processing vast amounts of damage data through screening criteria, they developed robust fragility relations for different pipe materials, using geometric mean PGV, angular distortion and lateral peak ground strain as IMs. Specifically for the gas distribution network performance, they comment that it remained almost undamaged, owing to the good

ductility of MDPE pipelines. Further, M. J. O' Rourke *et al.* [89] enriched the fragility relation proposed by M. J. O' Rourke [87] with four additional data points obtained from the 1999 Kocaeli event. This fragility relation does not differ significantly from the initial one, hence demonstrating that the latter is fairly stable. All strain-based fragility relations are plotted in **Figure 5**; all empirical fragility expressions cited herein are summarized in **Table 3**.

Lanzano *et al.* [69] published one of the few studies that addresses complete fragility curves,

Table 3. Summary of the most recent empirical fragility functions in terms of repair rate (RR/km) for buried pipelines found in the literature; PGV in cm/s, K_1 and K_2: correction factors that apply to certain pipe types, PGD in cm, ε_g: peak ground strain, $GMPGV$: geometric mean PGV

Reference	Fragility function	Remarks
M. J. O' Rourke and Ayala (1989)	$(PGV/50)^{2.67}$	Wave propagation damage
T. D. O'Rourke and Jeon (1999)	$0.00109 \cdot PGV^{1.22}$	Wave propagation damage, CI pipes
ALA (2001)	$K_1 \cdot 0.002416 \cdot PGV$	Wave propagation damage, various pipe typologies
ALA (2001)	$K_2 \cdot 2.5831 \cdot PGD^{0.319}$	PGD damage, various pipe typologies
M. J. O' Rourke and Deyoe (2004)	$513\varepsilon_g^{0.89}$	Wave propagation damage, segmented pipes
M. J. O' Rourke and Deyoe (2004)	$724\varepsilon_g^{0.92}$	Combined wave propagation and PGD damage, segmented pipes
M. J. O' Rourke and Deyoe (2004)	$0.034 \cdot PGV^{0.92}$	Wave propagation damage, surface waves
M. J. O' Rourke and Deyoe (2004)	$0.0035 \cdot PGV^{0.92}$	Wave propagation damage, body waves
M. J. O' Rourke (2009a)	$1905\varepsilon_g^{1.12}$	Wave propagation damage, segmented pipes
T. D. O'Rourke *et al.* (2014)	$10^{-4.52} GMPGV^{2.38}$	Wave propagation damage, CI pipes
T. D. O'Rourke *et al.* (2014)	$0.0839\varepsilon_g + 0.41$	Lateral ground strain damage, CI pipes
M. J. O' Rourke *et al.* (2015)	$2951\varepsilon_g^{1.16}$	Wave propagation damage, segmented pipes

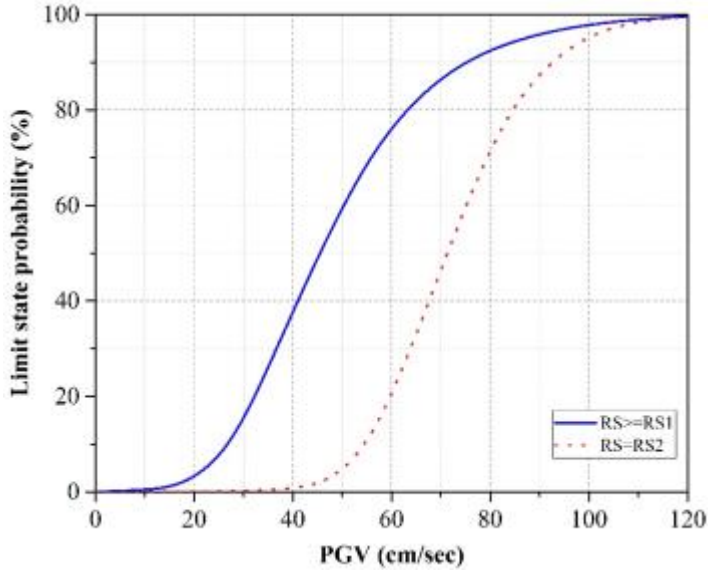

Figure 6. Seismic fragility curves for buried natural gas pipelines developed by Lanzano *et al.* (2013) (adapted from [69]).

in the sense of probability of exceedance of a specific performance level given some measure of ground motion intensity. Their investigation regarded continuous, steel-welded, natural gas pipelines subject to TGD and the IM used was PGV as well. Three discrete damage states were established: slight, significant and severe, which then were associated with corresponding risk states, according to projected estimations of environmental consequences. Utilizing a vast database of past earthquake damage, from which only the well documented cases were considered, seismic fragility curves were developed by fitting the useful data with a lognormal CDF (**Figure 6**). The extension of this work incorporates fragility curves due to PGD [70].

Identified gaps in the literature

It is evident that the available studies on seismic fragility of pipelines are in short limited to empirical expressions, which inherently are applicable to cases where ground motion and pipe characteristics are similar to the ones used to derive those expressions. Therefore, it appears unreliable to generalize them and incorporate them unconditionally into seismic risk assessment and mitigation methodologies and software. In light of this, analytical fragility curves, verified against experimental results, are expected to provide damage prediction capabilities under a wider range of seismic scenarios and for an extended typology of buried pipelines, also allowing for the consideration of special phenomena affecting pipe response, such as the spatial variability of ground motion and soil-pipe interaction. Further, there seems to be some bias in the available

damage information, as most of it concerns segmented water pipelines. Vulnerability research addressing continuous steel pipelines with welded joints, which is usually the case in buried natural gas pipelines, is scarce; hence, this issue remains to be illuminated.

6 Pipeline health monitoring for maintenance and rehabilitation

The demand by society imposed on the engineering community for sustainable infrastructure is constantly growing. To achieve the goal of sustainability, two major requirements must be met during the design life of an infrastructure: regular maintenance and quick rehabilitation after an extreme event. In this respect, an integral part of the desired service lifecycle of lifelines is the implementation of non-destructive Structural Health Monitoring (SHM) methods during their operation towards the reliable diagnosis of their structural condition. Several, yet not entirely different definitions have been proposed for the arguably fast-evolving practice of SHM. According to Chang [90], for instance, SHM provides the means to continuously gather (near) real-time information on the integrity of infrastructure without interruption of their service, with the final goal being hazard mitigation. Nevertheless, almost all definitions agree on some basic aspects [91] that are typical of SHM applications, including:

(1) Almost real-time health screening
(2) No service interruption during the monitoring process
(3) Deployment of sensing instruments capturing on a continuous basis variations in specific metrics that determine the state of the structure
(4) Transmission of acquired data through an established wired or wireless network
(5) Data analysis in order to detect damage patterns and assess damage modes and extent

SHM finds application on nearly every lifeline system and tends to become standard practice nowadays, given their importance for the societal well-being; underground energy pipelines are no exception to this. Past experience has shown that natural hazards such as earthquakes can cause severe damage to buried natural gas pipelines, leading to content leakage, which in turn may trigger explosions, fires and atmospheric pollution. On top of this, pipe deterioration may be accelerated by previous time-dependent material degradation and ageing, or even manufacturing defects. Therefore, it becomes clear that pipeline monitoring to track structural integrity over time should be a matter of priority for natural gas pipeline operators in the framework of a long-term management strategy that ultimately aims for life extension of the pipeline and minimum

supply interruption. Besides, the pipeline industry is bound to special regulations that require the implementation of inspection procedures on existing pipelines [92]. Pipeline SHM techniques can prove useful both (a) as a prevention tool, in that they can detect in-time accumulated damage due to service loads, wearing and pre-existing flaws prior to any failure and (b) as a remediation tool, in that they can rapidly localize and characterize incurred damage immediately after the occurrence of an earthquake.

Scheduled maintenance by means of visual in situ inspections has now been replaced to a great degree by cutting-edge techniques that not only offer a broader insight of the structure's integrity indicators both in space and in time, but also minimize labor and downtime costs. Excluding the outdated and inefficient in situ inspection, three are currently the main sensing technologies used in pipeline SHM [93]:

(a) in-line inspection techniques
(b) fiber optic sensing
(c) remote sensing

Of the three, the first two are the dominant trends in pipeline industry, and for this reason emphasis herein will be placed on these.

In-line inspection techniques

Perhaps the most widely adopted approach in SHM of buried natural gas pipelines today is the so-called in-line inspection. Essentially, small autonomous devices known as 'smart pigs' (the term 'pig' derives from Pipeline Inspection Gauge) and carrying sensors, data recorders and transmitters are inserted inside the pipeline and driven by content flow, 'in-line' with it. As they travel long distances in the interior of the pipe, the mounted sensors obtain continuous measurements of various parameters, depending on the desired inspection tasks; these are typically related to geometry checks, strain analysis, metal loss and crack detection. In this manner, large pipeline segments can be examined at reduced times without blocking the transportation process of natural gas.

The basic principle behind the measuring activity that gives meaning to the obtained data is that consecutive measurements are taken over time, thus any change with respect to previously obtained values related to undamaged state will denote a health issue. After proper statistical

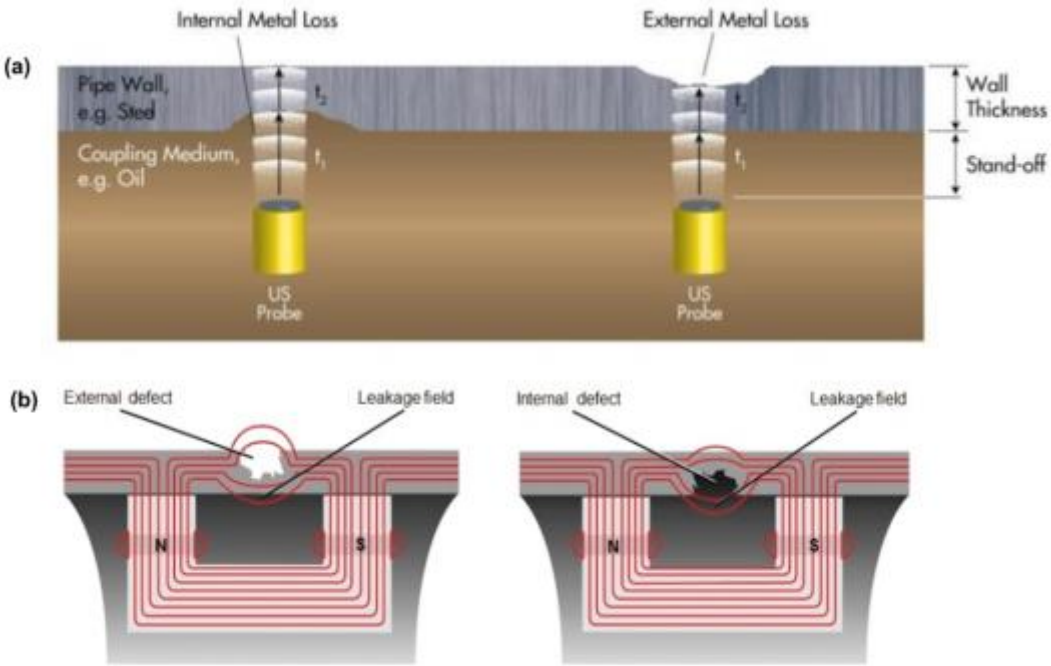

Figure 7. Schematic views of in-line inspection technologies: (a) Principle of ultra-sound based sensors (reprinted from [92]; (b) Principle of magnetic flux leakage sensors

processing, these data are compared to measurements corresponding to the so-called 'learning' period and diagnosis is then made with respect to the integrity of the pipeline.

Commercially available in-line inspection tools are based on various sensing technologies [93]. Among them, ultrasound-based sensors are common in the market for metal loss and crack inspections. These are sensing transducers that emit ultrasonic pulses in the direction of the pipe wall. The acoustic signals are then reflected from both the inner and the outer wall surface and captured back from the transducer (**Figure 7a**). From the knowledge of the sound velocity in the medium and by measuring the traveling times of the signal, wall thickness is computed and any metal loss can be inferred. The transducers may be piezo-electric or electro-magnetic, with the latter being the case for natural gas pipelines as the former require a liquid medium to function, and may also be installed on the external surface of the pipeline. Another highly popular in-line inspection technology tailored to corrosion detection of steel pipelines is magnetic flux leakage. According to the underlying physical principle of magnetization, the inspection unit transmits magnetic flux into the pipe-wall, creating a magnetic circuit. If metal corrosion is present in certain regions, there will be some sort of leakage in the magnetic field, which is detected by

magnetic sensors placed on the unit (**Figure 7b**). Moreover, the latest industry trends suggest the combined utilization of different sensing technologies on a single in-line inspection tool in order to carry out more reliable, multi-purpose pipeline inspections.

Distributed fiber optic sensing

Fiber optic sensors are one of the most promising technological developments in the field of SHM, although their first use can be traced back as early as the 1970s [93]. The function of fiber optic systems is based on the physical properties of light propagation: the goal is to associate unexpected variations in the light signals as they travel along fiber strands with damage patterns. Through various configurations, fiber optic sensing offers diverse capabilities in measuring a number of different parameters, including strain, temperature, pressure and acceleration [91]. What is of interest in examining the condition of a pipeline subject to earthquake effects is primarily the strain levels in the pipeline. Discrete and, lately, distributed fiber optic sensors have been used for strain monitoring purposes. Although discrete sensors provide unmatched resolution and accuracy in local-scale measurements, they are not suitable for global monitoring, as this would require the installation of thousands of them along the pipeline, together with a complex wiring system, leading to prohibitively high costs.

This significant drawback is surmounted by the distributed fiber optic sensors, which are capable of efficiently monitoring large portions of such elongated systems as pipelines. Distributed sensors are fairly simple in their structure; they comprise a single silicon fiber cable sensitive at its whole length, which is tightly bonded to the pipe wall upon installation in order to allow lossless transfer of the material strains. Low attenuation levels ensure that distributed sensors perform well over distances of up to 25 km [94]. Other advantages of the distributed sensing technology include simple cable connections to the data receiver and reduced installation effort and cost.

Distributed fiber sensing technology relies on one of the following three optical effects: Rayleigh scattering [95], Raman scattering [96] and Brillouin scattering [97]. Technical details about these fall out of the scope of this study and may be found in the relevant references. Brillouin scattering-based implementations are usually the method of choice, since they suffer the least from signal losses and they are capable of long-range monitoring [98]. Several experimental studies have been conducted that demonstrate the effectiveness of the method. For

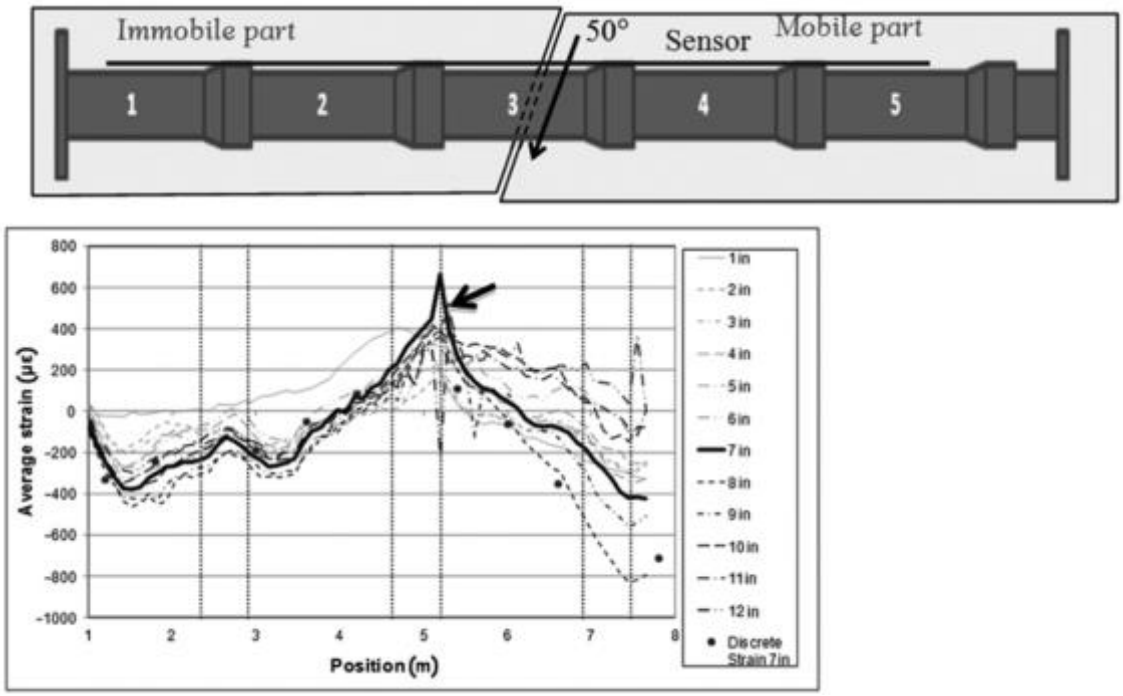

Figure 8. Select results from the pipe strain monitoring experiment with distributed Brillouin sensors conducted by Glisic and Yao (2012), indicating detected damage (reprinted from [98])

instance, Inaudi and Glisic [94] present the results of the field application of a previously developed Brillouin distributed strain, temperature and combined strain-temperature sensing instrument (DiTeSt) [99]. Excellent performance of distributed strain monitoring on a buried gas pipeline subjected to landslide loading was reported, as well as successful detection of the leakage spot by the distributed temperature sensors during a gas leakage simulation. In an earlier laboratory test, Ravet *et al.* [100] took advantage of the unique capability of distributed Brillouin sensors to measure both tension and compression at the same time, in order to detect the starting point of buckling in a steel pipe under axial compressive load. To ensure prior knowledge of the location of buckling initiation, weakening of the specimen wall was performed at a specific region. Comparison between the measurements from the distributed Brillouin sensor and installed strain gauges along the pipe body showed good agreement, and tensile strains were successfully detected by the distributed Brillouin sensor, signifying the initiation of the buckling process. Glisic and Yao [98] put extensive efforts in developing an integrated damage monitoring method of buried concrete segmented pipelines exposed to seismic effects, using distributed Brillouin scattering-based fiber optic sensors. In validating the method with large-scale testing, PGD was simulated to act on a 13 m-long pipeline assembled inside a test basin

and covered with soil, while strain readings obtained from the fiber optic sensors were verified against data from conventional strain gauges (**Figure 8**). Damage accumulation in the joints was mainly observed, as expected, and the sensing system achieved to identify these patterns as strain peaks in the strain profiles. The applicability of the method can be safely extended to continuous steel pipelines according to the authors.

Critical summary and issues to be addressed

The aforementioned pipeline inspection techniques are not universally applicable in industrial practice, as they present specific drawbacks that limit their implementation. A crucial factor that determines the suitability of in-line inspection tools is the potential of the pipeline to permit passage of the pig unit through its body (known as 'pigability'), which depends on a number of pipeline attributes, such as the size of the pipe section, the operational pressure and the flow conditions [101]. Besides, in-line inspection requires some degree of manual operation, as well as efficient energy management of the wireless sensors. More importantly, in-line inspection techniques are considered less suitable than distributed fiber optic sensing for emergency-state rapid damage detection following an earthquake, as they require longer operating times. On the other hand, fiber optic solutions are particularly expensive, and their cost tends to increase dramatically with higher measurement accuracy. Distributed fiber sensors also require more intricate installation procedures and ensuring of good bonding with the pipe wall is a prerequisite for accurate sensor readings; further to this, optimized placement of the distributed sensors on the pipe circumference is another concern for reliable integrity monitoring [102].

As general remarks concerning the full spectrum of available inspection technologies, it should first be underlined that there is a general difficulty in handling effectively the vast amount of data that are acquired from long-term pipeline monitoring facilities, and this may place doubts on the credibility of the results. To this end, efforts should be put towards the development of efficient data processing tools that incorporate sophisticated threshold-based algorithms of deterministic or statistical background, in order to reliably interpret captured metrics variations on the basis of previous samples. Second, the major challenge is to take advantage of the existing pipeline SHM technologies in a holistic approach involving rapid post-rupture health assessment, fast repair actions and decision-making in the direction of network resilience. Such considerations should not ignore the fact that, during a post-earthquake crisis period, power

supply and wireless communications networks may experience long-lasting outages, hence hindering any integrity assessment works.

7 The emerging concept of resilience in pipeline networks

Resilience is a recently developed and rapidly-accepted concept in the field of lifeline engineering that can be understood in the context of emergency situations caused by natural (e.g. earthquakes, floods, hurricanes) or man-made (e.g. vehicle collisions, bomb explosions) extreme events that induce abrupt variations in the performance of lifelines. In engineering terms, resilience denotes a highly desirable property referring to either physical (infrastructure) or social (communities) systems that requires multidisciplinary considerations for its quantification (it draws information from seismology, earthquake engineering, economics, social and management sciences) and careful treatment to discern it from closely related concepts such as vulnerability, fragility, risk and sustainability.

Analytical treatment of resilience

In a pioneer work, Bruneau *et al.* [103] set the foundations for the quantitative assessment of seismic resilience. They define a resilient system as one that obeys to three basic rules:

- It is characterized by enhanced reliability.
- It generates tolerable levels of losses when experiencing failure.
- It is capable of returning quickly to a previous performance standard after failure.

The preliminary identification of these core features of resilience helps develop a comprehensive mathematical definition of it. Drawing upon a study by Cimellaro *et al.* [104], seismic resilience is an index R representing the capacity of an infrastructure system or community to withstand earthquake effects by retaining an acceptable level of performance over a given post-earthquake time period, through a process involving loss estimation, collection of resources, relief strategy planning and restoration actions. The time-dependent performance of the system is measured with functionality $Q(t)$, a dimensionless function of time denoting the service quality of an infrastructure system at any time instant as a proportion of the full

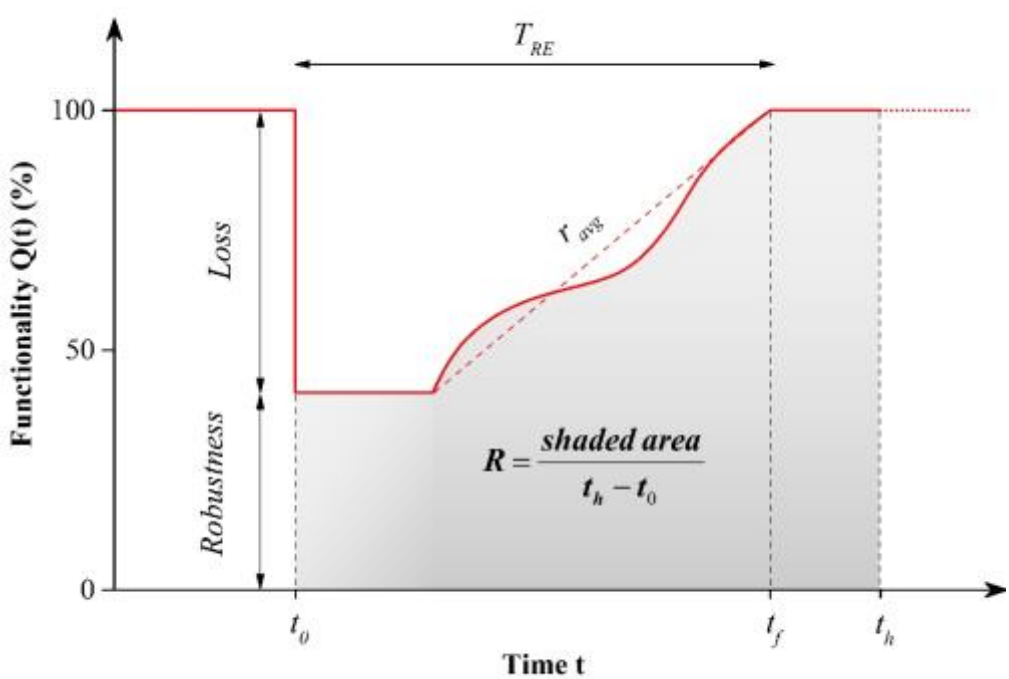

Figure 9. Graphical illustration of time-variant functionality, resilience, rapidity and robustness; R: resilience; r_{avg} : average rapidity; T_{RE} : recovery time

functionality corresponding to the initial, intact state of the system, assumed to be equal to 100% (**Figure 9**). Functionality can be modelled as a non-stationary stochastic process [104]. A mathematical definition of seismic resilience under the consideration of a single seismic event is possible:

$$R = \frac{1}{t_h - t_0} \int_{t_0}^{t_h} Q(t)\,dt \tag{21}$$

where t_0 is the time of the occurrence of the seismic event and t_h is the investigated time horizon. Graphically, Eq. (21) represents the shaded area underneath $Q(t)$ over the time interval $t_h - t_0$, normalized with respect to this interval, as illustrated in **Figure 9**. The mathematical definition of functionality involved in Eq. (21) may vary depending on the system examined.

Dimensions of resilience

While establishing a quantitative definition of resilience is difficult, evaluating it and finding ways to enhance it pose a further challenge, given the various sources of uncertainty that arise and the subjectivity of the problem. This task can be facilitated if the idea of resilience is broken down into simpler descriptors, as Bruneau *et al.* [103] propose: (a) rapidity, (b) robustness, (c) redundancy and (d) resourcefulness. Detailed descriptions of these quantities are available in Refs. [103,105]. Rapidity and robustness are quantifiable (refer to **Figure 9**), while redundancy and resourcefulness are more abstract qualities of an infrastructure system, difficult to quantify and also interlinked with each other. To make a distinction from rapidity and robustness, redundancy and resourcefulness provide the means to achieve resilience, whereas rapidity and robustness are descriptors of the final outcome.

Bruneau *et al.* also introduce an alternative way to qualitatively characterize resilience depending on its context. On this basis, they distinguish the technical, organizational, social and economic dimension of resilience. As it becomes clear out of this classification, different performance measures need to be employed for the quantification of these four dimensions, varying from case to case. The first two dimensions of resilience are considered at the facility/physical system level, while the latter two refer to the wider community spectrum.

Previous research on seismic resilience assessment of buried pipeline networks

Only a handful of studies on quantitative evaluation of seismic resilience are on record, since it is a topic that has gained popularity lately. Esposito *et al.* [66], in assessing the impact of the 2009 L'Aquila earthquake on the local gas network performance, analyzed the over time network functionality reinstatement described in terms of the ratio of reconnected customers over total customers after the shock, showing that only 40% of the customers were reconnected to gas supply after two months. This functionality evolution was compared to a hypothetical ideal one, which however is not explained how it was estimated. The low restoration level achieved is attributed to the fact that reconnection was permitted only to buildings that received a green-tag during post-earthquake inspection.

More recently, Cimelarro *et al.* [106] developed and applied to a case study a comprehensive quantitative framework for the seismic risk analysis of gas distribution networks considering the impacted network functionality and the ensuing recovery process. Network functionality defined as a function of the time-variant gas flow and total operating pipe length was computed with numerical modeling using SynerGEE software. A single seismic scenario was extracted by de-aggregating local hazard maps corresponding to 22% probability of exceedance in 50 years and median PGV maps were calculated using attenuation relationships. Only PGD-induced pipe damage was considered and fragility relations developed by ALA [83] were exploited to express pipe damage distribution in a repair rate format. Then, 14 pipe breakage scenarios were defined and localized within the network based on a Poisson probability model, bridge collapse hypotheses and engineering judgment. The study ended with the estimation of resilience indices of the gas network for all damage scenarios, before and after the application of a retrofit alternative including emergency shutoff valves and flow dividers.

The need to extend and expand the seismic resilience assessment framework to buried natural gas networks

Despite the valuable but scarce previous contributions, the relevant literature database is still lacking an integrated risk assessment and management methodology for buried natural gas networks that would address the involved aspects altogether: seismic hazard, seismic vulnerability of individual network components, interdependence of network components, overall network vulnerability and seismic losses and comparative evaluation of loss mitigation strategies reflected on measured resilience levels, in a pre-earthquake probabilistic context and under the consideration of the particularities of such a network, such as the weak flow

redistribution capabilities and the high fire likelihood due to gas escape. The development of such methodology would assist engineers in accomplishing optimized network designs in terms of resilience in earthquake-prone areas; it would also assist managerial groups in (a) decision-making at a post-earthquake level aiming at limiting consequences and regaining initial network functionality shortly and (b) in enhancing preparedness levels at a pre-earthquake level in order to deliver the most effective emergency response in case of a future seismic event. So far, most of the risk-related research has focused on loss assessment and loss reduction resulting from different policies; however, a comprehensive seismic risk mitigation framework (and probably software tool as well) centered at seismic resilience, as the one described herein, would expand far beyond. Evident is also the lack of a robust and reliable functionality metric tailored to natural gas networks, that would account for the dependency of other lifelines on the gas network [107].

8 Current codes of practice and guidelines for earthquake-resistant design of buried pipelines

Presently, only few modern norms worldwide dictate requirements for the protection of underground natural gas pipelines against seismic risk, the most notable being Eurocode 8 - Part 4 [16] in the European Union, the American Lifeline Alliance guideline [29] in the US and the design recommendations by Japan Gas Association [56]. In this section, the key points in each normative document are discussed.

Eurocode 8 provisions (2006)

Part 4 of Eurocode 8 [16] provides a broad regulatory framework for the seismic design of pipelines, inter alias. According to it, the ultimate limit state of a pipeline is associated with structural collapse. Yet, it is implied that certain critical components of the system susceptible to brittle failure may be checked for a state prior to total failure. A two-level serviceability limit state hierarchy is prescribed; the lower one requires that the system remains fully operational and leak-proof and the higher one that it undergoes some level of damage without losing its whole supplying capacity. Another secondary safety hazard that should be taken into consideration in ultimate limit state design is explosion and fire in the event of an earthquake-induced breakage and the potential consequences on people and the environment. The determination of the seismic

actions should be based on the two principal sources of damage, i.e., seismic wave propagation and PGD.

Further, Eurocode 8 states that pipe inertial forces related to ground acceleration are of minor significance in comparison with the forces caused by ground deformation, thus they may be neglected; this simplifies the nature of the problem, converting it to a static one. With regard to wave propagation effects, Annex B of Eurocode 8 recommends the conservative method developed by Newmark [17] to determine the induced pipe strains and curvatures, as long as the soil is stable and homogeneous. As for the spatial variability in ground motion, no particular guideline is provided; however, in the chapter concerning above-ground pipelines, it is suggested that spatial variability is accounted for when the pipeline length analyzed is over 600 m or the ground is characterized by longitudinal non-uniformities. It is also noted that pipelines buried in dense soil are allowed to be designed solely for the effects of wave propagation.

When it comes to PGD, Eurocode 8 provides a set of specific design rules to improve resistance. In the case that the pipeline route crosses an active fault zone, the design should in general ensure maximum flexibility of the soil-pipe system, so that the pipeline can withstand larger deformations. Pipelines crossing strike-slip or reverse faults should be oriented in such an angle that the affected pipe segment is subjected predominantly to tension and not compression, thus taking advantage of the available ductility of steel. Other site-specific construction measures include minimizing the burial depth, increasing pipe wall thickness within a 50 m-zone on both sides of the fault, using a hard and smooth pipe coating to reduce the angle of interface friction between the pipe and the soil, using soft soil as backfill and avoiding significant deviations from a straight line alignment. Annex B suggests that a simplified method to modeling the phenomenon is to apply a relative static displacement at the point of pipe-fault intersection. To address the threat of soil liquefaction, increasing the stiffness of the system is a proposed measure, either by increasing the burial depth of the pipe or encasing it in stiff container tubes.

For steel welded pipelines, Eurocode 8 specifies that the maximum ductility of steel is not exceeded and buckling modes are not observed. For the first condition, ultimate steel tensile strain is set to 3%; for the second, the allowable steel compressive strain is proposed as the smaller value between 1% and $20(t/r)$, where t is the thickness and r the radius of the pipe.

American Lifelines Alliance guidelines (2001)

The report prepared by American Lifelines Alliance (ALA) [29] focuses on roughly the same points with Eurocode 8. Furthermore, it suggests performing three-dimensional nonlinear quasi-static finite element analysis for investigating PGD effects, considering soil-pipeline interaction. The mechanical behavior of both the pipe material and the soil mass should be modeled as inelastic. The length of the pipeline model has to be carefully selected in such a way that the imposed constraints at the ends do not produce unrealistic local axial deformations. The need to ensure a more refined mesh in the proximity of the PGD region is also highlighted.

As regards modeling of wave propagation effects, it is stressed that induced flexural strains may be neglected, due to being of considerably lower magnitude compared to axial strains. Moreover, the conservative assumption that soil strains are caused by surface waves is allowed to be adopted, since this results in larger strains. Wave propagation-induced soil strains are usually expected to be lower than 0.3%.

A list of performance criteria are proposed in Appendix A, which, however, are not universally applicable; different permissible values may be set for each specific case. For axial strains caused by PGD, two performance states are suggested: operable state and pressure integrity state. For the first, non-exceedance is dictated of a 2% tensile strain and a compressive strain defined as

$$\varepsilon_{c,cr} = 0.50\left(\frac{t}{D'}\right) - 0.0025 + 3000\left(\frac{pD}{2Et}\right)^2 \tag{22}$$

where $D' = \dfrac{D}{1 - 3/D\left(D - D_{\min}\right)}$

Eq. (22) is adopted by Gresnigt [62]. The corresponding limits for the second are $\varepsilon_{t,cr} = 4\%$ and $\varepsilon_{c,cr} = 1.76t/D$. Concerning the effects of wave propagation, the resulting bending stress must not exceed the yield strength of steel. The allowable tensile strain is set to 0.5%, while the allowable compressive strain is defined as $3/4$ of the limit specified in Eq. (22). All above limits are in force only on the condition that strict welding procedures are adopted during construction of the pipeline.

Appendix B provides the soil spring relationships presented in section 2.2. It is noted that the calculation of axial springs must be performed considering backfill soil properties.

Recommended practice by Japan Gas Association (2000)

"Recommended practice for earthquake-resistant design of gas pipelines" developed by Japan Gas Association (JGA) [56] constitutes a revised version of the initial guideline, issued in 1982. It features a strict methodology for designing high-pressure transmission pipelines to Level 2 seismic motions. Design seismic motions are specified based on two performance levels similar to Eurocode 8: Level 1 states that "operation can be resumed immediately without any repair", while Level 2 states that "the pipeline does not leak, though deformed". The design flow comprises two phases. In the first phase, the design seismic motion is determined considering the potential existence of active faults near the pipeline route, which may require a fault analysis. The second phase consists of a sequence of simplified formulas that estimate wave propagation-induced pipe strains. Specifically, the natural period of vibration T of the surface soil layer and then the apparent wavelength ℓ of the assumed seismic motion are calculated first:

$$T = 4H/\bar{V}_s \tag{23}$$

where H is the thickness of the layer and $\bar{V}_s$ the weighted S-wave velocity,

$$\ell = V_{app}T \tag{24}$$

where V_{app} is the apparent wave propagation velocity of the motion. Following is the calculation of the axial ground displacement U_h at the depth of the pipe axis as

$$U_h = \frac{2}{\pi^2} c \cdot T \cdot S_V \cos\left(\frac{\pi z}{2H}\right) \tag{25}$$

where c is the seismic zone coefficient, S_V the spectral velocity of the soil layer and z the pipe burial depth. The peak ground strain of uniform, regular ground can then be estimated:

$$\varepsilon_g = 2\pi U_h/\ell \tag{26}$$

The last step involves extraction of the pipe strain from the ground strain using a strain transfer coefficient

$$a = q\frac{1}{1+\left(\frac{2\pi}{\ell}\right)^2\frac{\mathrm{EA}}{k_a}} \tag{27}$$

where q is a coefficient accounting for soil-pipe sliding and k_a is the soil spring stiffness in the axial direction. Finally, pipe strain is calculated as

$$\varepsilon = \alpha \cdot \varepsilon_g \tag{28}$$

and checked against an allowable strain of 3%. The previous procedure applies to straight pipe segments, provided that no fault affects the pipeline; a slightly different last step is proposed for pipe elbows and tees.

Other directions in legislative or guideline texts

After a rigorous search in the literature, it was concluded that other standards and regulations provide hardly any additional useful information on the issue. The B31 Code for Pressure Piping by the ASME [108,109] highlights that the maximum axial stress for a restrained pipeline should be constrained up to a level that no buckling is caused; the permissible value for the sum of all the longitudinal stresses is established as

$$\sigma_{\alpha,\max} = 0.90\sigma_y \tag{29}$$

Axial strain should not develop further than 2%. Furthermore, design against for soil liquefaction and landslides should be performed on the basis of the operability performance level.

A relevant report by FEMA [53] states among others that, unless previously corroded or poorly assembled, buried pipeline systems are quite unlikely to get significantly affected by traveling seismic waves; on the other hand, permanent ground deformations are considered to have a higher damaging potential. Increasing the ductility capacity of the pipeline and ensuring protection against corrosion and high quality welding are qualified as capable measures to improve the performance of the pipeline even under large permanent ground movements.

Some observations

With the exception of JGA guidelines, existing codes are not seen to provide a concrete framework for earthquake-resistant design of buried steel pipelines, rather their utility is limited

to coarse tips and recommendations of construction practices and oversimplifying assumptions. Eurocode 8 insists on the conditionally reliable but outdated method by Newmark, disregarding the long recognized soil-pipe interaction influence on pipe response. ALA goes one step further by proposing soil spring relationships; their applicability, however, has limitations as noted in section 2.2. On the other hand, JGA does define a specific methodology for design of buried pipes solely against wave propagation, but this considers only axial response and homogeneous soil conditions; in addition, it does not address the question of what soil spring constants should be used.

Overall, all previous guidelines are far from comprehensive. More importantly, striking is the total absence of any citations on fragility analysis and seismic resilience of gas networks. Focus is unfairly given on the component level instead of the network level. Similarly, no information is provided on spatial variability in seismic ground motion and SHM techniques.

9 Discussion and conclusions

This review study presents and comments on the state-of-the-art in seismic analysis and risk assessment of buried steel natural gas pipelines through an integrated treatment of the most significant aspects, advancing into the emerging concept of resilience. The most important identified gaps or insufficiently addressed issues are the following:

- The true cyclic pattern of seismic excitation has been mostly overlooked in previous soil-pipe interaction studies; the same applies for the potential influence of kinematic interaction. Further research is also needed to explore the sensitivity of pipeline seismic response to variable horizontal soil stratification along the pipeline route. The latter is a matter of particular concern for buried pipelines considering their spatial dimension.
- Thorough investigations are required to shed light into the somewhat obscure effect of spatially variable ground motion on pipeline seismic response, especially under inhomogeneous site conditions. Experimental work on this is deemed necessary to verify the existing theoretical findings.
- Current knowledge on seismic fragility of buried steel pipelines need be expanded towards analytical damage rate expressions that will be derived from extensive parametric numerical analyses, in order to account for a variety of seismic scenarios, pipe and soil configurations.

This will also allow the examination of the effect of soil-pipe interaction and differential ground motion on pipeline damage rates.

- Two are the predominant pipeline SHM methods used in practice today: in-line inspection and distributed fiber sensors. Both have benefits and drawbacks in different perspectives, though fiber-based monitoring appears to be a more attractive option in the long term. The challenge here is to efficiently utilize them in post-earthquake health screening in order to maximize rapidity of recovery and consequently resilience levels.
- Seismic resilience of buried natural gas networks has not been studied adequately so far. Resilience levels depend on pipeline robustness, which in turn depends on pipeline fragility, which in turn may be dependent on various factors as mentioned previously. Resilience is also affected by rapidity, which can be improved through better SHM and emergency preparedness, and redundancy, which is generally not an inherent property of natural gas networks. All these interdependencies need to be rigorously scrutinized in order to proceed to the development of reliable seismic resilience assessment methodologies.
- Modern seismic standards and guidelines are still in an immature stage with respect to seismic resilience of buried natural gas pipelines, as there is not a single reference to it, probably due to the slow updating process. Furthermore, there is generally an obvious lack of a detailed design methodology that will safely guide the practicing engineer throughout the process of seismic design. In contrast, a set of empirical recommendations are mainly provided, which however are sometimes incomplete or outdated.

The above conclusions point to the fact that future research should be oriented towards adaptation of seismic resilience into the territory of buried natural gas networks with the aim of ensuring their long-standing, up-to-the-standards seismic performance. Efforts should also be expended into the quick incorporation of the latest resilience-related findings into contemporary seismic code provisions.

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