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Marie Skłodowska-Curie Actions

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Institutions Involved (Work Package 02, D2.1)

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2. BAUHAUS-UNIVERSITAET WEIMAR
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EXperimental & Computational Hybrid Assessment of Natural Gas Pipelines Exposed to Seismic Risk

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Table of Contents

1 Introduction

A preliminary stage of the EXCHANGE-Risk project requires to gather and classify information about the broader natural gas pipeline network operating in the European continent. This is a necessary step in order to evaluate the density of the network crossing seismically active geographical areas and possibly to develop seismic risk maps for natural gas pipelines in subsequent phases of the project. The database can also be used to easily extract technical data about pipelines needed for benchmark numerical studies in the framework of the project.

For this reason, an electronic database was created to:

(1) keep track of all major natural gas pipeline systems supplying the European continent and, at the same time,

(2) provide taxonomy capabilities in terms of various pipeline characteristics.

Table 1: List of fields defined in the table to reflect basic pipeline characteristics.

Field name	Data type	Description
Pipeline name	Short text	
Operator	Short Text	Full pipeline operator name.
Operating since	Date	Starting date of operation.
Current operating status	Yes/No	Operational or out-of-service.
Capacity	Number	In billion cubic metres per year.
Geographic characterization	Short Text	Transnational, intranational or intercontinental
Transit/Served countries	Short Text	List of sovereign countries that are crossed or served by the pipeline (sourced from another table).
Functionality	Short Text	Gathering system, transmission system or distribution system.
Installation setting	Short Text	Specify type of bedding (onshore buried, onshore aboveground, onshore elevated, offshore subsea).
Total length	Number	In kilometres.
Material	Short Text	Steel, PVC, PE, etc. (sourced from another table).
Diameter	Number	In millimetres.
Operating pressure	Number	In Megapascal.
Pipe segment connectivity	Short Text	For example welded joints, mechanical joints etc.
Notes	Long Text	Other notes related to the pipeline.

2 The database

The database was built in the Microsoft Access environment and its structure is straightforward; it consists of two objects, a table where data are recorded and a linked form to facilitate input of new table entries. The table currently contains 15 fields that provide fundamental *technical*, *operational* and *geographical* information on each pipeline. These fields have been selected on the basis of general availability and after consulting professional GIS data providers; they are summarized in Table 1. With a view to eliminating the possibility of inconsistent data input, appropriate data types have been assigned to each input field and specially designed tables/lists have been linked to specific fields in order to restrict the range of admissible field values.

As of now, the table contains 48 entries in total, both onshore and offshore pipelines. The main sources used to populate the database are the dedicated operator websites for each pipeline, hence data may be deemed reliable. Moreover, pipeline data have been crosschecked against larger pools providing pipeline information collectively [1–3]. However, not all pipeline features included in the database are readily available on the web (e.g. operating pressure), and for this reason some pipeline entries are not complete. In addition, some entry fields (e.g. installation setting) were postulated based on other fields, due to lack of explicit information. It is also important to mention that, in this premature form of the database, all pipeline systems registered in it are transmission pipelines. The objective is to extend this database in order to incorporate gathering and distribution pipelines within Europe as well, and possibly pipeline systems located in North America. The database is continuously updated. Some views of the database are provided in the following figures.

The database itself is available as an individual Microsoft Access file that is uploaded in the portal.

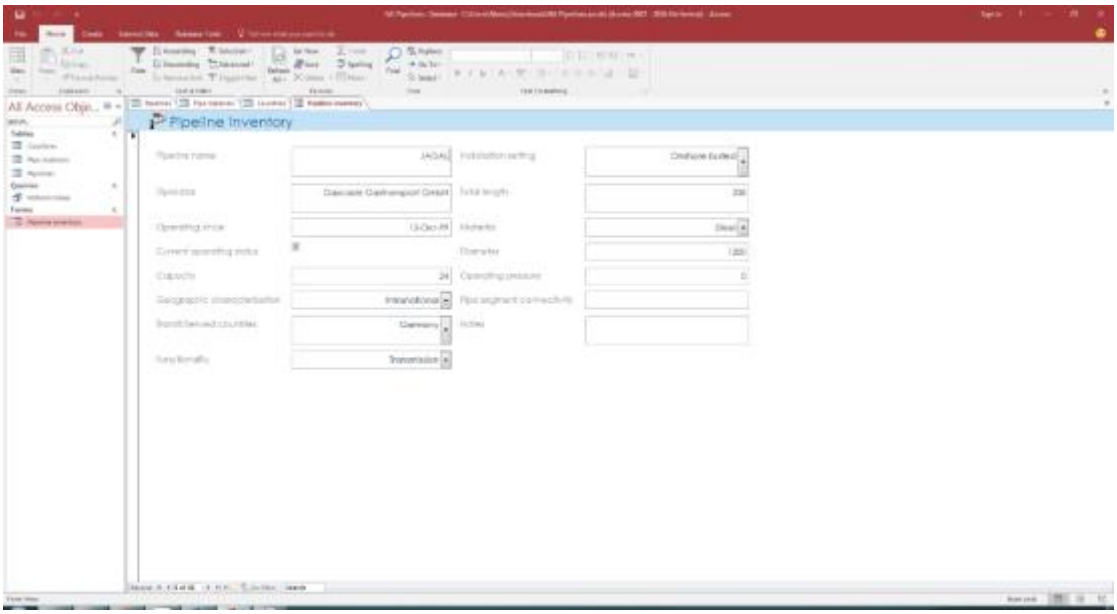

Figure 1: View of the database form used to insert new pipeline entries into the table.

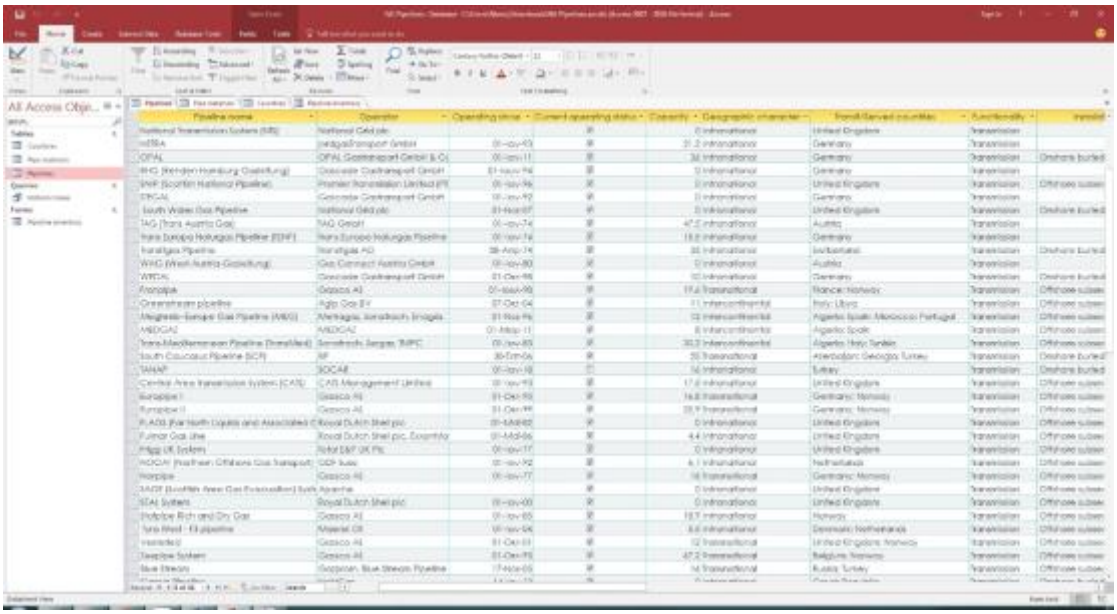

Figure 2: Partial datasheet view of the database table.

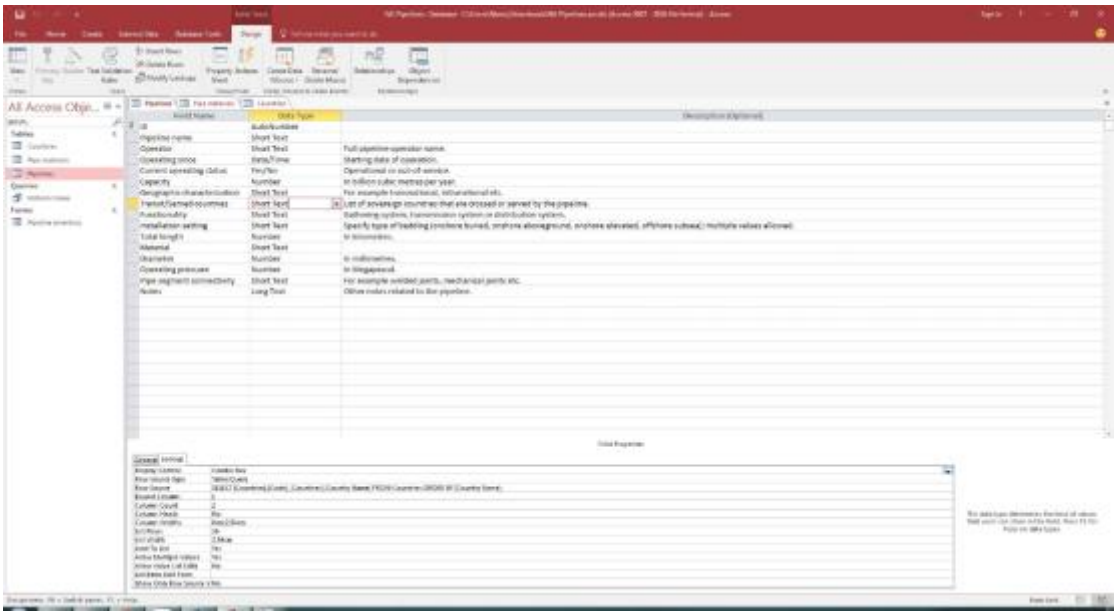

Figure 3: View of the database table in design mode; visible are the fields specified, along with the associated data types.

References

[1] ENTSOG (the European Network of Transmission System Operators for Gas) n.d. http://www.entsog.eu/ (accessed June 25, 2016).

[2] GIE - Gas Infrastucture Europe n.d. http://www.gie.eu/ (accessed June 25, 2016).

[3] Wikipedia contributors. List of natural gas pipelines. Wikipedia, Free Encycl 2016. https://en.wikipedia.org/wiki/List_of_natural_gas_pipelines (accessed June 25, 2016).

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Project Full Name: EXperimental Computational Hybrid Assessment of Natural Gas pipelines Exposed to Seismic Risk

Marie Skłodowska - Curie Actions

Deliverable # 2.2: Seismically induced hazards, failure modes and limit states and performance indicators of NG pipelines.

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

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Table of Contents

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.

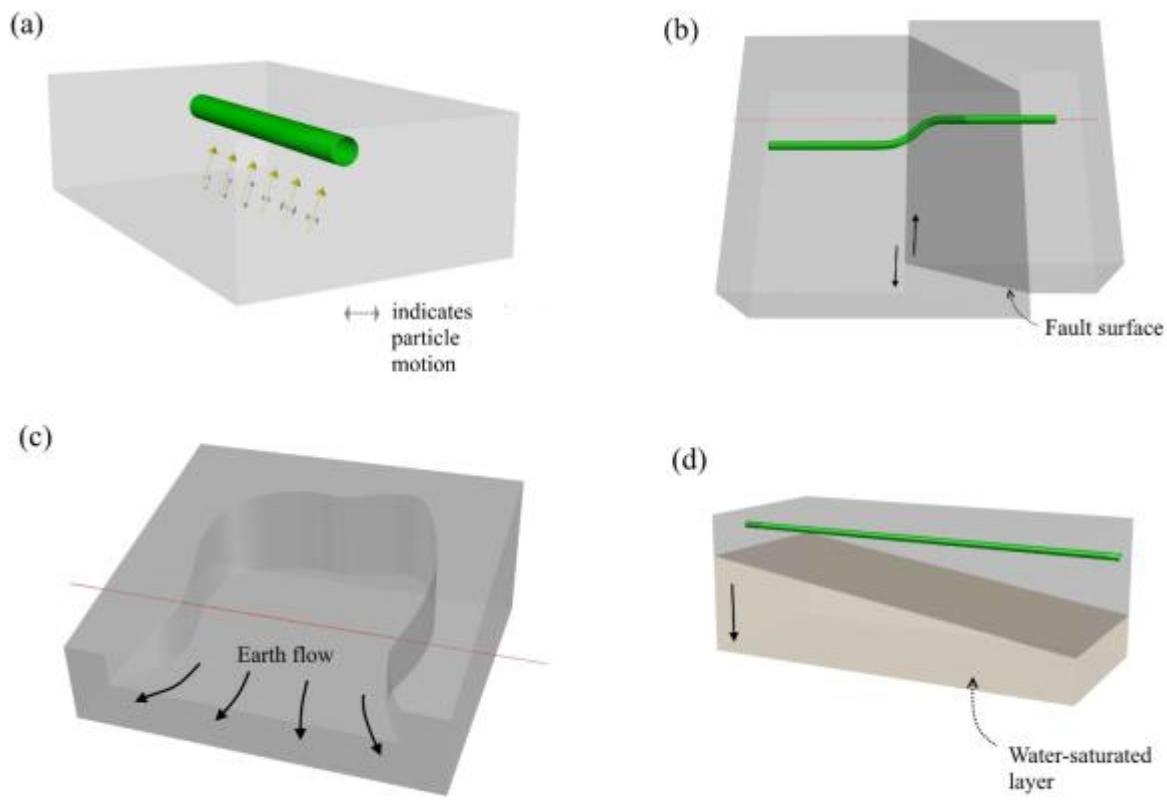

Figure 1. Illustration of the major earthquake-related geohazards threatening the structural integrity of buried pipelines; (a) seismic wave propagation; (b) strike-slip fault movement; (c) landslide in the form of earth flow; (d) liquefaction-induced settlement

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

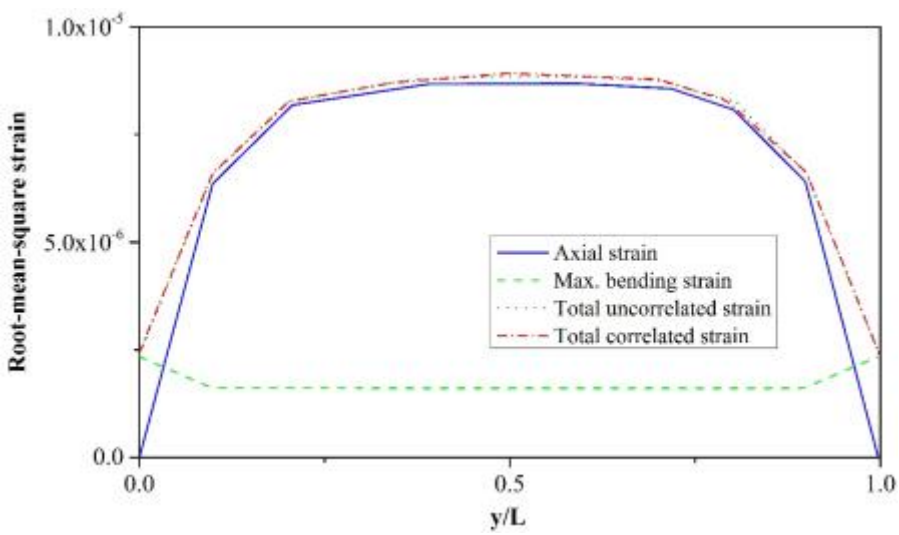

Figure 2. Distribution of root-mean-square strains along the pipeline axis for pipe orientation that coincides with the epicentral direction of the input motion, as calculated by Zerva (1993) (adapted from [47]).

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

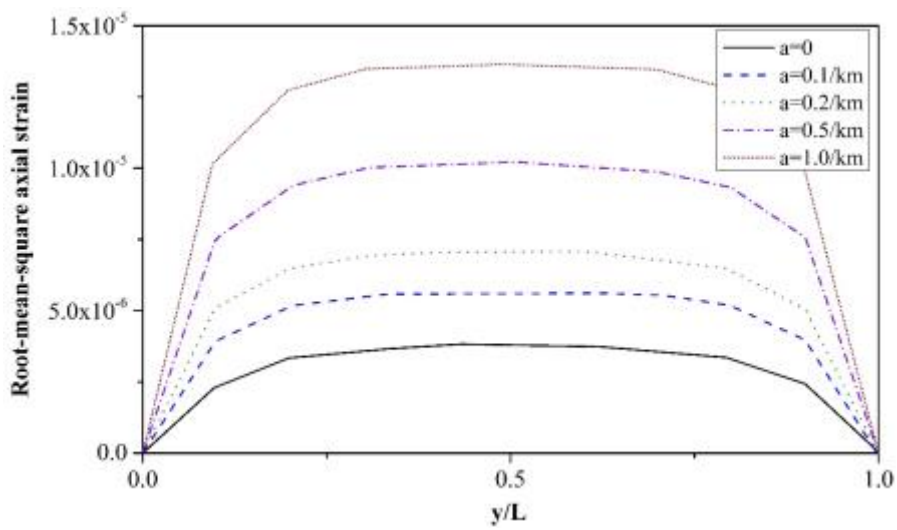

Figure 3. Distribution of root-mean-square axial strains along the pipeline for different values of the incoherence parameter, as calculated by Zerva (1993) (adapted from [47]).

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**).

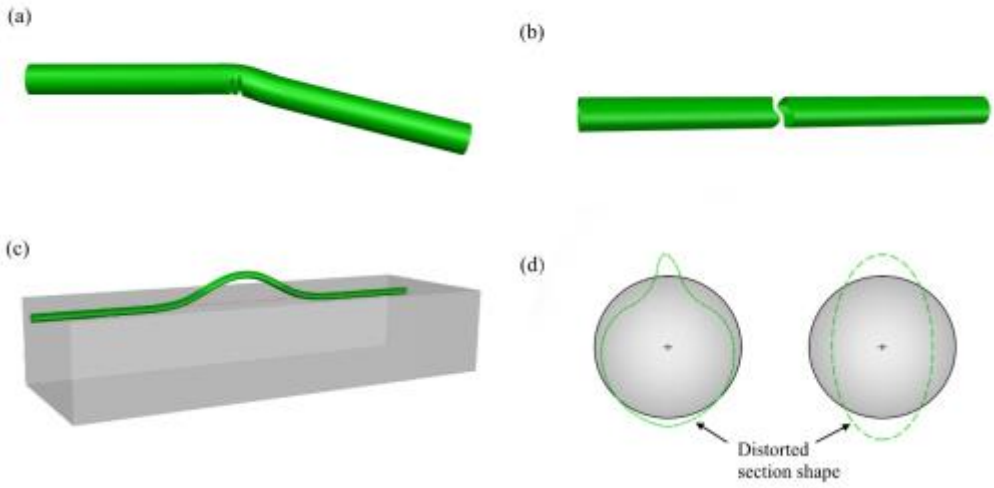

Figure 4. Frequently observed failure mechanisms in buried continuous steel pipelines: (a) local buckling; (b) tensile fracture; (c) upheaval buckling; (d) cross-section distortion

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

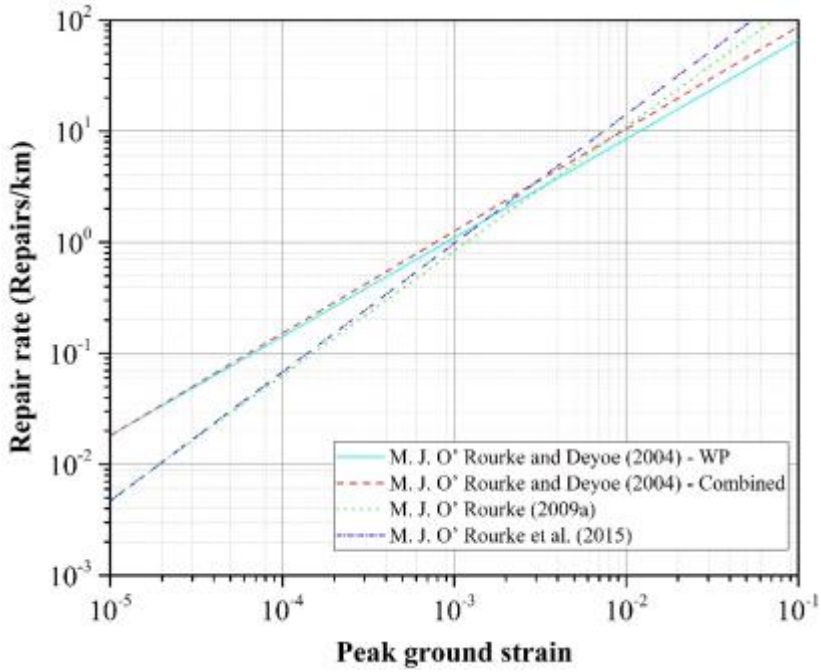

Figure 5. Comparative log-scale plot of published strain-based empirical fragility relations for buried pipelines.

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

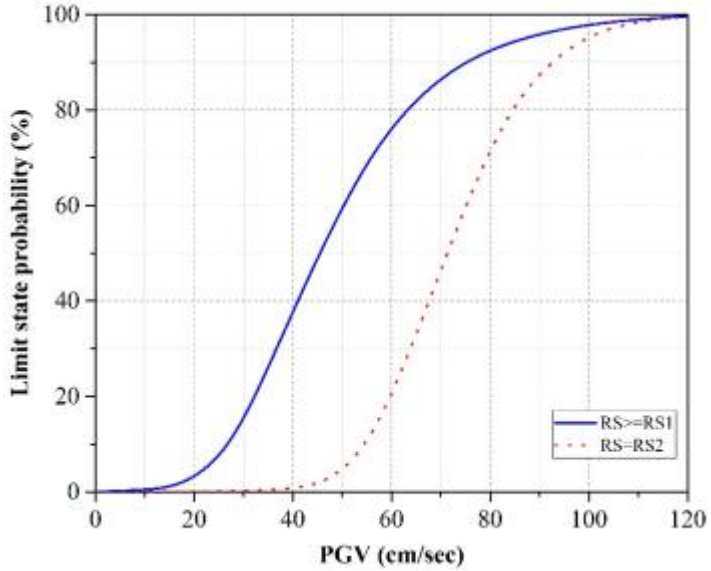

Figure 6. Seismic fragility curves for buried natural gas pipelines developed by Lanzano *et al.* (2013) (adapted from [69]).

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

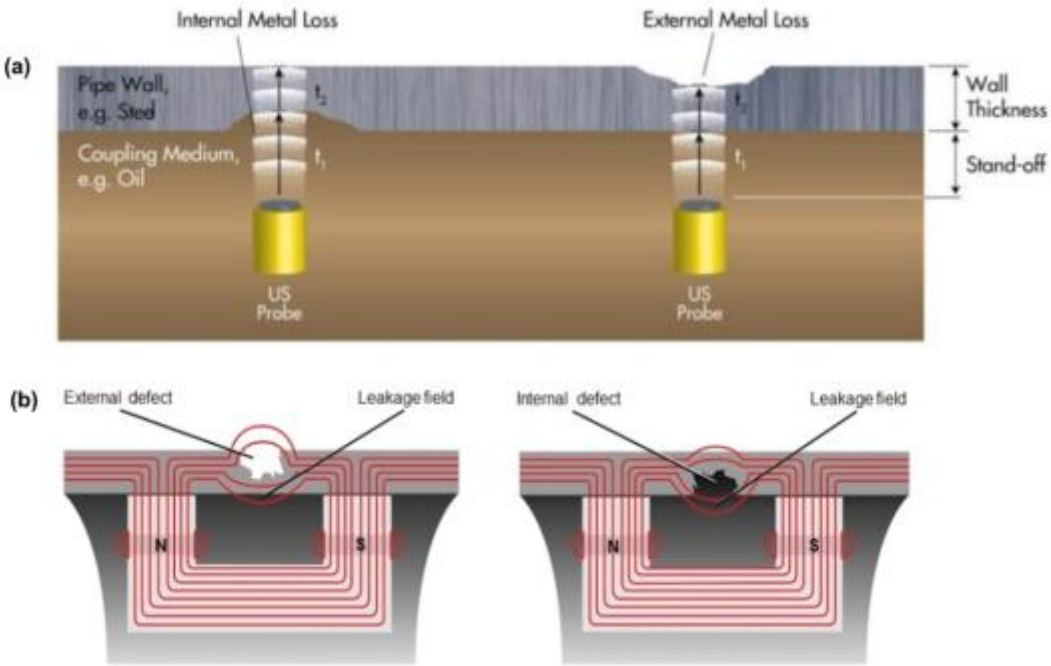

Figure 7. Schematic views of in-line inspection technologies: (a) Principle of ultra-sound based sensors (reprinted from [92]; (b) Principle of magnetic flux leakage sensors

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

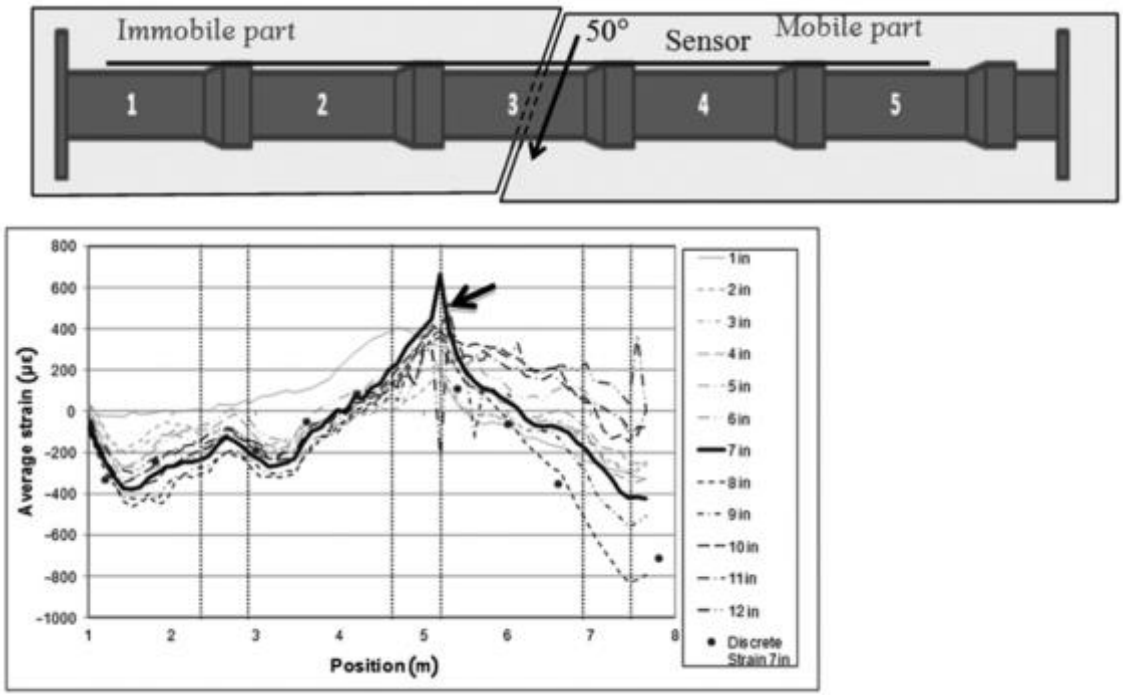

Figure 8. Select results from the pipe strain monitoring experiment with distributed Brillouin sensors conducted by Glisic and Yao (2012), indicating detected damage (reprinted from [98])

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

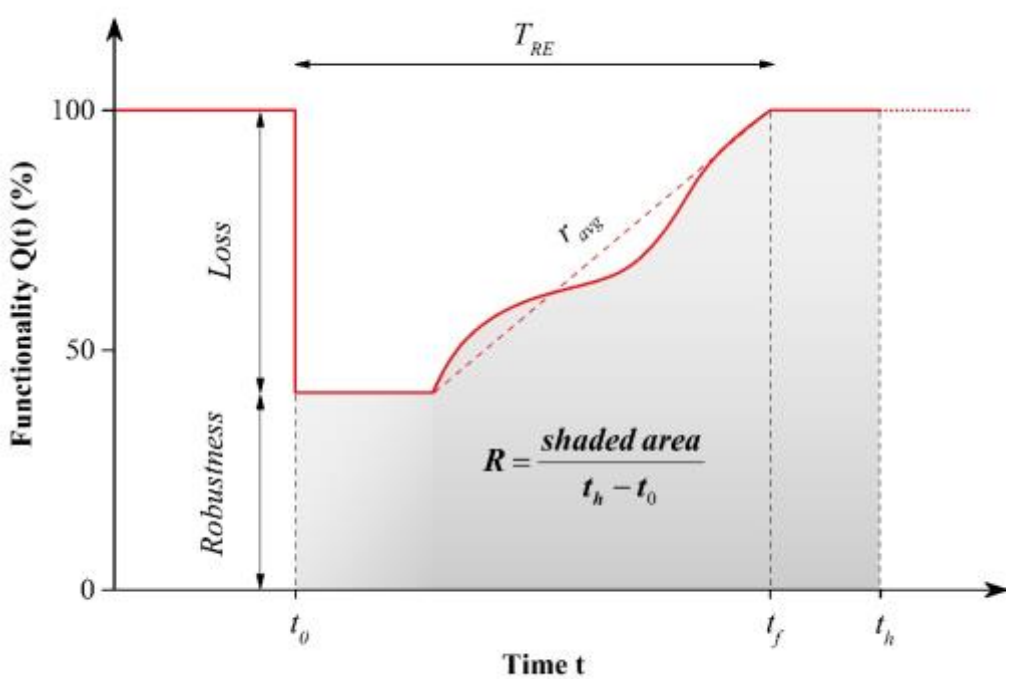

Figure 9. Graphical illustration of time-variant functionality, resilience, rapidity and robustness; R: resilience; r_{avg} : average rapidity; T_{RE} : recovery time

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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Inspection and monitoring for life-cycle management of natural gas pipelines

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Abstract

As natural gas pipelines consist significant lifelines for the industrialised society, their safety and reliability against various man-made and natural hazards at any point in time are of great importance. This review paper is an overview of typical earthquake-induced damage scenarios and failure modes of natural gas pipelines, as well as different state-of-the-art technologies that are currently implemented for inspection and monitoring. As different elements of a pipeline have various structural typologies, respective inspection and monitoring technologies are diverse as well; their choice being always related to the specific objective of identification.

Methodologies which use recorded data measured along a pipeline for operational control, structural failure risk assessment and life-cycle management are put into context with approaches of multiple criteria and multi-objective analyses that can be used in decision making, taking versatile technical, financial, environmental and societal aspects into account. The paper concludes with a critical discussion regarding the pros and cons of different inspection methodologies, limitations and challenges to be met.

1 Introduction

Natural gas pipelines are extensive lifelines that transport gas over distances of thousands of kilometres through regions with varying soil, site and geological conditions. The exposure of pipelines to various environmental threats such as humidity, chemical properties of surrounding materials, extreme temperatures and operational influence, contamination of gas and high pressure or fatigue loading can lead to health deterioration due to corrosion or damage. Typically these long-term processes can result in local failure, which becomes apparent as leakage.

Apart from deterioration that develops usually over long periods, pipelines can be affected and severely damaged by external events such as earthquakes, landslides, flooding, sea bed movement or man-made impacts (i.e., excavation injuries). In figures 1 and 2 the numbers of incidents that were identified as

significant by the U.S. Pipeline and Hazardous Materials Safety Administration (PHMSA) between 2005 to 2015 are shown for the U.S. gas distribution and transportation network.

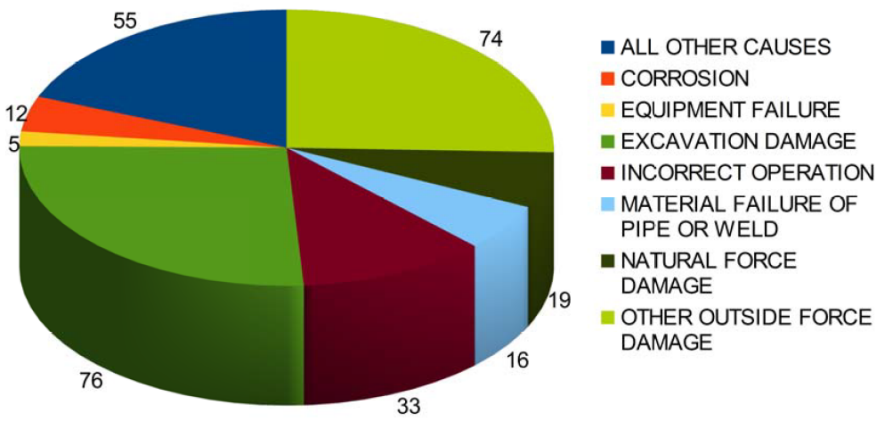

Figure 1: Serious incident rates and causes in the U.S. gas distribution network from 2005 to 2015: data source: U.S. PHMSA data base, 17th Oct. 2016

A comparison with older statistics [77], [5], [60] shows different distribution of the various causes of incidents, however, overall, the absolute number of significant incidents shows a decreasing trend.

The statistical data presented in figures 1 and 2 refer to the number of incidents neglecting the respective consequences. Recent studies have shown though, [51] that even though incidents caused by natural hazards are relatively rare, they cause 34 % of all property damage of pipelines as, for instance, pipe rupture can be far more severe than corrosion-induced leakage.

The majority of natural gas pipelines are expanded outside of urban areas, typically within a remote environment, thus hindering the reliable damage identification or tracing of the conditions that can lead to further deterioration. Some of the most essential requirements that need to be satisfied by respective equipment for large scale monitoring and inspection are:

- Acquisition of information about the pipeline's state over long distances without human intervention,
- High degree of resistance against harsh environmental conditions,
- Minimal interference of function of the pipeline and
- Sufficient precision of the measurements.

Data to be acquired by any technical device in-situ has to be processed and interpreted such that respective decisions about possible repair, interruption

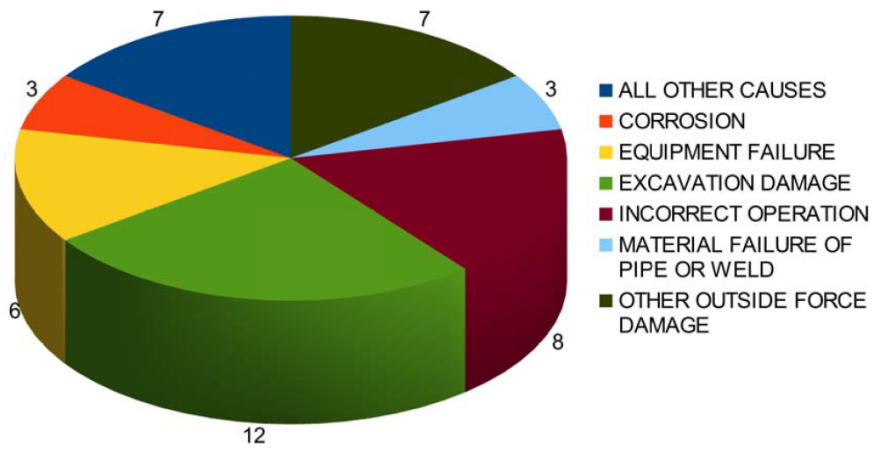

Figure 2: Serious incident rates and causes in the U.S. gas transportation network from 2005 to 2015: data source: U.S. PHMSA data base, 17th Oct. 2016

of service or maintenance can be derived. In this context, the main criteria to be addressed are the maximum probability of detection of a certain damage and the minimum probability of false alarm. Furthermore, tools have to be provided that allow the operators to decide within short time about the need for immediate shut-down and/or appropriate measures of repair. In this context, monitoring and inspection serves the optimisation of maintenance scheme.

Natural gas pipeline systems consist of several components that need to function interactively to provide a reliable and stable transportation of gas from the storage stations to the end-consumer. These components encompass:

- *Transmission pipes*: Mainline pipes are usually of 80-150cm in diameter, while pipes delivering gas to or from a mainline have smaller diameters of 15-60cm. Transmission pipes consist of carbon steel that needs to satisfy specific national and international standards. Highly advanced plastics are also often utilised in distribution networks.

- *Compressor stations*: To ensure that the gas remains highly pressurised during its transportation, periodic compression of the natural gas is required. These stations are commonly located at intervals of approximately 100 to 200 km along a line. Apart from being pressurised, natural gas is often dehydrated and filtered in compression stations.

- *Metering stations*: Between compressor stations, additional metering stations are designed along a pipeline to measure the gas flow.

- *Valves*: To allow the shutdown of sections of a pipeline for maintenance purposes, repair or replacement, valves are placed at distances of several

kilometres.

- *Control stations*: All data that is monitored along a pipeline is processed in central control stations from where the pipeline network is operated. For this purpose, data transmission lines are installed along the pipelines, to collect measurements from compression and metering stations. The components for the communication and data acquisition that provide the information about the service of a pipeline to the control stations form Supervisory and Data Acquisition (SCADA) systems.

In this review paper, several techniques for inspecting the structural integrity of gas pipeline systems are presented, involving long-term environmental and operational effects and natural hazards such as earthquakes or landslides, that can lead to exceedance of different damage states of the pipelines. After a summary of typical defects and damage modes, different technical solutions are discussed. Given that the data acquired by means of monitoring inspections is incorporated into risk assessment and risk management of pipeline systems, section 4 is dedicated to schemes and methods which developed to support informed decision-making by the gas network stakeholders.

2 Defects and damage modes to be detected in pipelines

Prior to developing a strategy for monitoring and inspecting a pipeline system, strength degradation and damage modes, as well as the respective severity of damage as per the operation of the system need to be first defined. Assuming normal operation of a pipeline network, there are different mechanical phenomena that can lead to the ultimate limit state. These phenomena are effectively targeted by appropriate inspection and/or monitoring techniques and are listed as follows:

- Corrosion,
- Third party (i.e., man-made) damage such as excavation,
- Fatigue and other cracks,
- Material and/or construction defects,
- Ground movement, e.g. due to landslides, flows and earthquakes,
- Leakage.

2.1 Corrosion

Corrosion is an oxidation of metal that is caused by chemical reaction of the material with a second element [31]. It has to be considered as one of the major

sources that causes deterioration in natural gas pipeline systems. In general it is distinguished between internal and external corrosion.

External corrosion is the chemical reaction of the steel pipe with its surrounding environment. In case of buried pipes, the surrounding material is soil. Submarine pipelines are surrounded by sea water when deployed on the sea bed.

As natural gas can be contaminated by small amounts of water or other corrosive substances, corrosion can be provoked from inside a pipe as well. This process is called internal corrosion.

In both cases, the thickness of the pipe is reduced and this results into loss of strength and subsequent cracking, leakage or, ultimately, rupture. Therefore, the respective detection methods mainly rely on the identification of changes of the wall thickness.

Detection and mitigation of pipeline corrosion is a subject on which very much research and industrial development has been carried out in the previous decades. Several studies have been described in the literature, focusing on the description and numerical modelling of corrosion [38], [72], [42], its detection [49], [53], [14], [13] as well as its evaluation and reduction, for both the cases of internal and external corrosion [45], [77], [21], [38], [69] and [70]. Several of these topics are further covered in more general guidelines and publications such as [5], [7], [19] and [31].

2.2 Excavation damage

In case that the soil covering a gas pipe is excavated without due consideration, damage can be triggered ranging from insuring the corrosion protection of the pipe to rupture. In the U.S. alone, 1630 pipeline incidents due to third-party excavation were reported by PHMSA [60] within 1993 to 2012. Note that third-party refers to excavations that were not carried out by the operators of the pipeline or the contractors. This is also supported by figures 1 and 2, where excavation remains one the major sources of damage of buried pipelines. Comprehensive discussions over excavation damage can also be found in [60] and [77].

2.3 Ground movement

Ground movement can be caused by a number of different phenomena, the most hazardous of which are landslides and earthquakes. These actions can cause large (generally static or slowly developing) deformations that can subsequently lead to severe induced strains and rupture. Depending on the pipeline construction technique (i.e., continuously-welded or segmented pipes), the respective failure modes are classified into two groups as presented below [55] based on the reported evidence from the literature and the associated failure criteria.

2.3.1 Damage to continuous steel pipelines

Based on the assumption of a flawless welding process and corrosion-free conditions continuous, welded steel pipelines, five primary, ground movement-induced failure mechanisms are commonly distinguished [84], [55]

- *Pure tensile rupture*: Excessive longitudinal strains due to axial tension can result into pipe rupture. This type of failure has been rarely observed in arc-welded steel pipelines with butt connections as they behave in a ductile manner, however, steel pipelines assembled with gas-welded slip joints are more vulnerable to this failure mode as they have very low tensile capacity. Examples of this failure have been observed after the 1994 Northridge earthquake [56] among others. According to [35], the ultimate tensile strain of X-grade pipe steel at fracture may reach 6%. Nevertheless, in engineering practice a more conservative value of 3% or 4% is applied. For the numerical description of the nonlinear structural behaviour of steel under tension, a suitable material model such as the Ramberg-Osgood model [64] is required.

- *Local buckling*: This failure mode is commonly refered to as *wrinkling*. It is a failure condition that occurs due to structural instability in a pipe under longitudinal compression, often combined with bending. Depending on the extent of the acting forces, this local distortion of the pipe wall can cause further bending and eventually tearing of the pipe. Local buckling has been observed in several pipelines after earthquakes [55]. In most cases, local buckling distortions were accumulated at the regions of geometry transition, such as bends and elbows. For design purposes, a failure criterion for pipe local buckling was proposed in [29] based on experimental studies, defining a critical compressive strain by means of the ratio of a pipe's wall thickness to its diameter. As stated in [55], this criterion is better applicable to thin-walled pipes, but is relatively conservative for thick-walled ones.

- *(global) Beam (upheaval) buckling*: Pipes can also be considered as long slender structures that are prone to global stability failure under longitudinal compression. This geometrically nonlinear behaviour usually results in a global deformation which does not necessarily lead to fracture and is therefore considered as less catastrophic [55]. Beam buckling can usually be prevented by a sufficient cover of overlying backfill soil. In fact, it has been shown [48] that there is a proportional relationship between buckling load and trench depth, i.e., if a pipeline is constructed at a depth larger than a critical cover depth, then local buckling will occur before beam buckling and vice versa. This practically implies that a minimum cover depth of 0.5 to 1.0 m, which is usually satisfied in practice, is adequate to prevent beam buckling, an assumption that has also been proven valid [48].

As beam buckling does not necessarily interrupt the gas flow, its identification requires careful inspection as reported, for example, in [47].

- *Flexural failure*: Due to the high ductility of steel pipelines, flexural failure hardly ever develops. On the contrary, a number of buried gas and liquid fuel pipelines were found after the 1971 San Fernando earthquake to have absorbed approximately 2.5 m of soil displacement in the transverse direction [57].

- *Section distortion*: Severe bending of a pipe can force the pipe to ovalize in a way similar to tunnel ovalisation under seismic excitation, thus risking pipeline serviceability. As ovalization can be quantified as a reduction of the pipe diameter along the direction of the shorter axis, the value of 15% diameter reduction has been identified as the respective threshold to avoid section distortion [28].

- *Damage to pipelines with welded slip joints*: While failure criteria for butt-jointed pipelines are mainly related to pipe material strength, for pipelines with slip, riveted or gas-welded joints, failure criteria have to be formulated with respect to joint strength, since the joints are usually weaker than the main pipe body. A number of studies involved the estimation of the strength of slip joints with inner and outer weld [76] in terms of joint efficiency, namely the ratio of joint to pipe strength. Joint efficiency values in the range of 35-40% [50] and [8] were obtained in all cases. Detailed damage evidence at welded joints exists for the case of the 1971 San Fernando earthquake, where most of the failures were observed at the welds of gas-welded joints.

A more in-depth discussion on the failure modes of continuous steel pipelines as well as recent observations of damage, can be found in

2.3.2 Damage to segmented pipelines

For segmented pipelines linked by means of mechanical jointing or fitting techniques (e.g. bell and spigot, flanged pipe joints), as may be the case with cast iron, ductile iron and concrete pipes, damage due to permanent ground motion appears to be mostly localized at the joints. For instance, following the 1991 Costa Rica earthquake six distinct failure modes occurring in segmented pipelines were identified in field inspections [54]:

- *Axial pull-out at joint*: This failure is most common when the tensile forces in the pipe barrel exceed the shear load capacity of the joints. As a failure criterion with respect to the onset of leakage at the joint-pipe interface in [22] was proposed to limit the joint slip along the contact area to the half of the joint insertion depth. The validity of this threshold value was confirmed by laboratory tests on concrete segmented pipes

connected with rubber gasketed joints subsequently [9]. In a more recent numerical and experimental study, considerable allowable longitudinal and rotational deformations were identified for ductile iron water pipes with bell and spigot joints [83].

- *Compressive 'telescoping' at joint*: Compressive failure of the bell and spigot joint pieces has been observed in several cases of severe compressive ground strains [3]. Based on laboratory tests [9] the ultimate compressive force of the concrete, obtained as the product of the compressive concrete strength and the pipe cross-sectional area (was suggested as joint crushing failure criterion for concrete pipes.

- *Disconnection at T-joint*: Tensile forces in one branch that is connected to a T-joint may lead to a slip and disconnection if the shear load capacity of the joint is exceeded.

- *Blowout at T-joint*: If the material strength of a T-joint is exceeded due to restraint forces and/or moments, the fitting body can break.

- *Break in union piece*: As the union piece connecting two pipe sections longitudinally creates an abrupt change in stiffness, this location becomes vulnerable especially under strong flexure. Accordingly, rupture can be provoked at the interface between union piece and regular pipe section under the action of large forces or bending moments.

- *Pipe segment break*: If the stresses in a regular pipe section exceed the strength of its material, cracks can be generated that can eventually result to fracture.

It is worth noting that most of the above failure mechanisms reported for continuously welded and segmented pipelines can in fact develop under permanent ground movement, the latter induced by natural hazards and their subsequent forces. However, as pipelines are also affected by corrosion, construction imperfections and man-made damage, the observed failure may well be the outcome of a number of simultaneously acting phenomena. As a result, health monitoring and inspection have to interpret the coupled effect of multiple phenomena on the serviceability and safety of pipelines in time [4]. The main technologies for inspection and monitoring of such systems are presented below.

3 Technologies for inspection and monitoring of pipeline systems

To ensure continuous function of natural gas pipelines over their lifetime, the control of deterioration and damage evolution is essential. Along these lines,

regular inspections and permanent monitoring of pipeline systems consist an important part of the lifecycle management of pipeline networks.

Monitoring of natural gas pipes includes sensing systems that register quantities determining the operation, mainly, flow velocity, pressure and humidity of the gas. Sensing systems are also utilised to assess and control structural integrity. The latter group of structural health monitoring systems include some fundamental requirements [26], namely, (a) nearly real-time health screening, (b) no service interruption during the monitoring process, (c) continuous capturing of variations in specific metrics that determine the state of the structure, (d) transmission of acquired data through an established, secure and sustainable wired or wireless network and (e) data analysis aiming to detect and assess damage patterns, location and extent. It is also noted that current SHM techniques do not only offer a broader insight of the structure's integrity in space and time, but also minimize labor and downtime costs. For this purpose, in case of buried and under water pipelines, operation and maintenance management almost completely relies on automated monitoring.

3.1 Modern technical tools for the inspection and monitoring of natural gas pipelines' state

Apart from the economical interest of pipeline operators, safety and environmental aspects are of major importance for these systems due to the major consequences of their potential failure.

Therefore, a permanent control of the structural integrity of natural gas pipelines is essential what lead to the development of respective national and international guidelines and regulations, the pipeline industry is bound to [6].

As pipelines are to their largest extent buried underground or located on the sea bed, the accessibility for visual inspections is very limited. Traditional methods of pipeline inspection include *hydrostatic testing* and *direct assessment* [7]. For a hydrostatic test, a section of a pipeline is filled with pressurized water, such that its pressure exceeds its operation one. Hydrostatic testing is able to detect flaws larger than a critical size. This procedure requires an interruption of operation and is expensive. It requires the acquisition, treatment and disposal of the water.

Direct (i.e., field) assessment, is based on a detailed investigation of critical locations along the pipeline, that are deemed most likely to suffer from corrosion. For these investigations several available techniques can be applied. In case of buried pipelines this can also require digging. Based on models and observations made locally, further measures are prioritized for additional inspection, rehabilitation or replacement [7], [49].

The afore mentioned methodologies require high technical expertise, considerable effort, are costly and may require the interruption of the gas flow. Consequently, three major sensing technologies have been introduced in the last decades to control the structural integrity of pipelines over long distances

[10]:

- In-line inspection techniques,
- Fiber optic sensing and
- Remote sensing.

Apart from these three groups of sensing technologies, several other inspection techniques for local inspections of pipelines are available. The length of observation along a pipe is in these cases usually too short for the monitoring of long distance pipelines. However, for smaller sections as, for example, within compression stations or in other industrial facilities these methods can be often applied very successfully. Therefore, some of these techniques are also mentioned here.

Each of the three groups of sensing technologies were developed with specific objectives. In-line inspection tools use different measurement instruments that serve, for instance, the identification of irregularities of pipe geometry, measurement of corrosion proxies such as wall thickness and the detection of cracks and leaks. With fibre optic sensors, strains and temperatures can be measured to detect leakage and to identify pipe deformations.

Remote sensing monitoring systems on the other hand, refers to different techniques that are based on optical methods and use data recorded by systems that are usually carried by moving ground or air vehicles. In the context of pipeline damage detection these techniques are applied, for example, to detect leakage, to prevent excavation damage by third-parties or to identify external sources that could jeopardize the safety of the pipe such as uncontrolled vegetation. The description of the different optical methods is beyond the scope of this article, however, a discussion of several optical methods is made in [71] and references therein. Technologies that allow the identification of changes of the ground surface above a pipeline, indicating planting of new trees, construction or other activities, by means of optical recordings with camera systems installed on air vehicles are presented in [30].

As already noted, all the above technologies have their strengths and limitations, hence, their appropriateness for a specific case depends on the pipeline characteristics and the requirements of the operator.

3.2 In-line inspection techniques

Probably the most widely adopted approach in structural health monitoring of buried natural gas pipelines today is the so-called in-line inspection (ILI). Essentially, autonomous devices known as *smart* or *intelligent 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 high speeds without blocking the transportation process of natural gas. An overview about the application of ILI technologies is, for example given in [34], while a more comprehensive state of the art of commercially available ILI technology can be found in [13] and [10].

In order to assess the state of a pipeline health repetitive and regular inspections need to be carried out. In this light, the combination of inspection data with numerical models provides the basis for decisions with respect to necessary maintenance and rehabilitation measures.

Modern intelligent pigs can carry devices based on various sensing technologies [34], [10], [13]. The mainly used measurement principles are listed below [13]:

3.2.1 Magnetic flux leakage (MFL)

This very well established technology is based on the magnetic saturation of the ferromagnetic pipe wall. Powerful permanent magnets installed on the ILI tool create magnetic fields in the pipe wall so that a magnetic circle is created between the magnetic yoke system installed on the ILI tool and the pipe. The profile of the magnetic field in a flawless pipe wall is expected to be smooth and linear. Internal and external metal loss disturbs the magnetic flux density, which is a function of the pipe wall's cross-sectional area such that the magnetic field leaks out of the pipe surface.

Depending on whether the magnetic sensors are mounted on the ILI tool circumferencially or in direction of the longitudinal axis of the pipe, they can detect flaws with an orientation along the pipe axis or in circumferential direction, respectively. By MFL inspection, any kind of metal loss in a pipe wall, caused by internal or external corrosion, erosion or any mechanical action, can be identified.

A more comprehensive discussion on the technical details of MFL is made in [14]. In [53] the performance of ILI with MFL technology is verified by means of field measurements.Various parameters determining the quality of the results of an MFL inspection were identified and addressed.

3.2.2 Eddy Current (EC)

Unlike MFL technology, the EC testing method is not based on changes of a magnetic field with constant intensity but uses alternating electrical currents. In a driving coil an alternating primary magnetic field is produced. By means of mutual inductance, a flow of ECs in the surface of the neighbouring pipe wall is caused. Accordingly, a secondary magnetic field is generated in the pipe wall.

Anomalies such as flaws caused by corrosion create a change of the EC's flow direction which affects the mutual inductance. A resulting change of

amplitude and phase between input and output is registered by means (of a second, receiving coil. This principle provides a highly sensitive tool to identify metal loss [13]. Uncertainties in the identification can be further reduced by combined application with MFL inspection [13] with reference to [73].

3.2.3 ElectroMagnetic Acoustic Transducers (EMAT)

Electromagnetic Acoustic Transducers (EMAT) are especially suitable for detecting stress corrosion cracking in the pipe wall and to identify disbondment or defect of protective coating. The measurement principle is based on the propagation of guided waves through the pipe wall. An impulse is generated by an arrangement of a coil in an electromagnetic field that forms an electroacoustic transducer. The energy travels then as mechanical (acoustic) wave through the pipe wall to a receiver which is located at relatively short distance. In case of a crack between emitter and receiver, part of the energy will be reflected and returned to the receiver that can also work as a receiver.

As coating is attenuating the acoustic wave, defects in the coating can be identified by analyzing the amplitudes of the received waves. Analysis of frequency, time-of-flight and wave modes allow for the distinction between cracks and other faults as well as for quantification of the crack size [13].

3.2.4 Ultrasonic testing (UT)

Ultrasonic inspection units can be used to measure the pipe wall thickness and to detect cracks in the pipe wall [13].

The measurement principles are based on the measurement of the time-of-flight of waves at very high frequencies. An ultrasound impulse is sent out by a transducer that works both as emitter and receiver. The sound wave is reflected first at the inner and subsequently at the outer wall surface. By determining the times when the two reflections arrive at the transducer and using knowledge about the sound velocity in the material of the pipe, the wall thickness is computed and any metal loss can be inferred.

For the detection of cracks in the pipe, the transducers need to be mounted to the ILI tool such that the emitted waves meet the inner surface of the pipe wall at an angle of 45°. If the sound wave reaches a crack on the pipe wall, part of the energy is reflected. Information about location, size and orientation of cracks can be therefore derived from the received signals containing the surface entry reflection as well as external and internal crack echoes. Depending on the direction in which the transducers are mounted on the carrier, cracks with longitudinal or circumferential orientation can be identified.

The transducers for UT inspection 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.

3.3 Distributed fiber optic sensing

Fiber optic sensors consist one of the most recent technological development that has been successfully used to monitor civil infrastructure. According to [10], first applications were reported in the 1970s. In most cases, the measured quantities are strain or temperature. However, today, fiber optic sensors are also integrated in transducers that measure acceleration, pressure or forces [26].

One of the most widely used fiber optic sensors are fiber bragg gratings. Similar to electrical strain gauges, they allow for local strain measurements only at those positions along a fibre where Bragg gratings are integrated into the fibre. The number of bragg gratings that can be used in a single fibre is limited to about 15 to 25. This limits the applicability of this sensing technique and makes it inappropriate for the global monitoring of a pipeline over long distances.

Distributed fiber optic sensing is a more suitable technology. The sensors basically consist of a single optical fiber cable that extend over a measurement range of up to 25 km [32] or more [4].

In pipeline monitoring, there are different ways to install distributed fiber optic sensors depending on the specific measurement task. For distributed strain measurements, the fibers need to be directly attached to the pipe wall. Several options, to implement this in practice are described in [4]. For the detection of leakage in buried pipelines, it is also possible to lay the fiber cables into the backfill at a short distance to the pipe surface. When leakage occurs, this will lead to a temperature change in the surrounding material of the pipe which can be in turn identified by distributed temperature measurements.

Distributed fiber optical sensing technology relies on one of the following three optical effects: Rayleigh scattering [61], Raman scattering [36] and Brillouin scattering [37]. Technical details about these fall out of the scope of this study and may be found in relevant references. However, some general remarks with respect to the application of these measurement principles to the monitoring of natural gas pipelines are given in the following paragraphs.

3.3.1 Rayleigh scattering-based sensing

There are two different types of Rayleigh scattering-based sensor technologies distinguished: distributed acoustic sensing and distributed disturbance sensors [4].

Rayleigh scattering based distributed acoustic sensing is sensitive to the fiber propagation conditions following external vibrations such as strong impulses. Therefore, Rayleigh scattering based distributed acoustic sensing can be used, for example, to monitor damage caused by third-parties. Maximal measuring ranges are reported to be between 40 and 50 km [4]. However, the spatial resolution depends on the measuring range. One advantage of Rayleigh scattering based-distributed acoustic sensing is the high sampling rate of tens of kHz [4].

For Rayleigh scattering based distributed disturbance sensors the best performance is limited to measurement ranges of some tens of metres according [59] with a very good spacial resolution, however. Accordingly, these measurement principles are not very well suited for the application to long pipelines.

3.3.2 Raman scattering-based sensing

Raman scattering occurs due to the change of magnitude of the molecular vibrations of the fiber material [24], [25]. As these vibrations are strongly influenced by temperature, Raman scattering distributed sensing can be applied to measure environmental temperature. Accordingly, Raman scattering-based monitoring is an appropriate technology for the detection of leakage [15], [4].

3.3.3 Brillouin scattering-based sensing

Brillouin scattering-based implementations have the advantage that they are capable of long-range monitoring [27], [63]. Typical measuring ranges lie between 30 to 100 km [4], [15]. Different measuring principles exist based on Brillouin scattering which are sensitive to changes of both temperature and strain.

Several experimental studies have been conducted that demonstrate the effectiveness of the method. For instance, the results of the field application of a previously developed Brillouin distributed strain, temperature and combined strain-temperature sensing instrument are presented in [32] and [78].

Combining Brillouin scattering-based measurements with Raman scattering-based sensing which is only sensitive to temperature allows for a distinction between effects due to strains and temperatures, respectively, without the need of installing a second fibre that is not connected to the specimen such that it is only measuring temperature [4].

In an earlier laboratory test, it was taken 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 [39] referring to [65], [66] and [85].

Extensive efforts in developing an integrated damage monitoring method of buried concrete segmented pipelines due to seismic effects, using distributed Brillouin scattering-based fiber optic sensors has been described in [27]. In validating the method with large-scale testing, permanent ground deformation was simulated to act on a 13 m-long pipeline assembled inside a test basin and covered with soil.

For distributed fiber optic sensing systems, typical strain resolutions of 20 $\mu\varepsilon$ at a varying spacial resolution are reported in literature [39], [15], [4]. The values given for temperature resolution vary between 0.01 and 1 K, depending on the methods used.

Further applications of distributed fiber optic sensing to the monitoring of pipelines are described, for instance, in [15], [24] and [39]. One critical issue with respect to the practical application of fiber optic sensor systems to pipeline monitoring is related to installation. Even though different methods such as integrating the optical fibers into special tapes and tubes [32] or textiles [39] have been already developed, the installation of the sensors during construction with heavy machinery and under harsh environmental conditions remains a challenge. Several proposals to solve these problems as well as further details referring to technical aspects with respect to different types of optical fibers and their application are given in [4]. There are also proposals to place distributed fiber optic sensors into the soil underneath or above a buried pipe such that temperature changes caused by leaking gas or fluid can be detected.

3.4 Local sensing techniques

While the inspection and monitoring techniques described in the two previous sections are most suitable for the application to transmission pipes, there are also other parts of a pipeline system such as compressor stations where the requirements FOR monitoring and inspection are different. In these cases, the length of the pipes are shorter, diameters are varying along the length, the pipes are partially accessible while in other instances they are buried in others.

Given the above distinct features of pipeline systems within the compressor stations but also the similarities between these stations and other industrial facilities, techniques that were developed for chemical and other plants are also applicable there. In the following, several of these techniques are briefly described.

3.4.1 Guided wave monitoring

One methodology that has been developed for the detection of cracks or corrosion pits uses ultrasonic guided waves. For the inspection of a pipe, ultrasonic waves are generated at one location and are transmitted along the pipe to both sides of the source along the longitudinal and circumferential direction. Anomalies in the pipe reflect these transmitted waves and send back signals from which information about the distance from the excitation point and attenuation can be extracted. An overview about and an introduction into this technology and its application to the inspection of pipelines are given, for example, in [43], [33] and [67].

Guided wave monitoring has been suggested especially for cases in which a pipe cannot be directly accessed for visual inspections as it is the case for insulated, buried or underwater pipes. Respective studies have been published, for instance, regarding very hot [82], underwater [52] and buried [81] pipes. From a single excitation point, typically ranges of up to 50 m in each direction (100 m in total) can be investigated depending on conditions influencing the

attenuation such as degree of corrosion, coating and surrounding material [43].

One critical aspect related to the above methods is that inspections with guided waves require a respective qualification of the operator as there are many factors influencing the quality of the results such as the choice of the respective modes [43], [52]. Therefore, research is concentrated on the development of methodologies that improve the identification process by applying sophisticated algorithms such as singular value decomposition or component analysis [44], [20], [41].

3.4.2 Acoustic methods

Acoustic monitoring techniques are commonly applied to pipelines for leak detection. They typically use specific sensors to detect continuous acoustic emissions generated by a leak source and propagated through the pipe as acoustic waves. The idea of using microphones placed on the pipe wall along a pipelie for the detection of leakage was patented in 1992 [16]. Nowadays piezoelectric sensors [62], MEMS or fiber optical sensors can also be used for the identification of acoustic emission [58].

A successful laboratory study on a branched system of PVC pipes has been presented in [58]. In another experimental study, acoustic emission sensing was applied to detect cracking of concrete in a segmented model pipeline [62].

However, the application of acoustic emission to the detection of leaks in pipelines is limited by the requirement of numerous sensors as the emitted acoustic waves attenuate with increasing distance from the location of the source. Further, it has to be mentioned that the leak must generate acoustic emissions at a level that allow a clear distinction from background noise [71].

3.5 Comparative assessment of alternative monitoring techniques

The discussed pipeline inspection techniques are very versatile. They were developed with different objectives with respect to defects to be detected and properties of the pipeline sections to be monitored. Accordingly, none of the presented techniques is universally applicable in industrial practice due to their specific limitations in implementation.

For example, one crucial factor that determines the suitability of in-line inspection tools is the potential of the pipeline to permit passage of a 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 [68]. Besides, in-line inspection requires launch and receive facilities. The operation depends on some degree of human intervention, as well as efficient energy management of the wireless sensors, transmission and storage systems. More importantly, in-line inspection techniques do not provide continuous information about the pipeline's condition and are therefore less suitable for emergency-state rapid damage detection.

Distributed fiber optic sensors on the other hand, require much more intricate installation procedures. Further, the installation on an existing buried pipe would require a complete excavation of the pipeline. Installation of fiber optic sensors, as well as any other permanent sensors needs to be performed during the construction process. This means that both the procedures of construction and sensor installation need to be coordinated very carefully. Methodologies and devices to protect the relatively fragile measurement equipment are essential.

4 Pipeline monitoring for operation support and risk management

Decisions in operation and management of pipelines rely to a great extent on data that has been measured at different locations and transferred to the control centre. However, this does not only require the recording and transfer of measured data to a control station but also processing of a large amount of data with respect to specific objectives.

The motivation to record specific quantities can be very versatile and so is the data processing techniques adopted as well as their respective objectives. While some of the measurements serve the control of the gas flow, others provide information about the state of the transmission pipe and further components of a pipeline. Two are the major objectives:

- Operation of the pipeline to ensure provision of natural gas according to the current demand of the customers and
- Risk management to guarantee a safe and cost-efficient long-term operation of the pipeline.

The first objective concerns mainly the tasks of a dispatcher controlling the operation of a pipeline while the second is more related to maintenance and strategic decisions.

4.1 Support of pipeline operation by monitoring

The natural gas transported through a pipeline is, as mentioned earlier, pressurized in compression stations. Because the consumption of gas at the end of the pipeline is varying depending on the demand of the end users, the fluctuations caused need to be adjusted by regulating the pressure in the pipeline. This implies that it is the task of a dispatcher to control the compression stations in such a way that the current linepack level of natural gas in the pipeline satisfies the request of gas by the customers at the time when the gas reaches the end of the pipeline, which inevitably, requires a high degree of expertise

An automated Gas Pipeline Operations Advisor (GPOA) has been proposed by [74] and [80] to support the dispatcher who may be less experienced or come

into stress situations during excessive demand or potential crisis management. Based on pressure and flow measurements, simulations of the gas flow are performed. The results of the simulations are then used by an expert system that applies rules derived from heuristic knowledge of senior pipeline operators such that the dispatcher receives recommendations with respect to the control of compressors.

4.2 Life-cycle and risk management

Safety and reliability are the fundamental prerequisites for the operation of a pipeline. Therefore, a reliability-based life-cycle management, as for example Frangopol [23] suggested for highway bridges, would also be sensible to be applied for the case of natural gas pipelines. One component of such a life-cycle management scheme is the monitoring of the considered system. Consequently, a respective monitoring system needs to be designed such that it can optimally provide the data which is required within a reliability-based life-cycle management scheme, a topic that has been addressed, for example, in [79] where a methodology has been suggested and applied to an offshore wind turbine structure that can also be adapted to natural gas pipeline systems.

Like other infrastructure systems, also many natural gas pipelines have already approached their design life time. Owners have then to decide if they can continue operation or if their pipeline has to be abandoned. To ensure safe and reliable operation, regular inspections and continuous monitoring can be implemented, a procedure that is also accepted by respective national regulations as pointed out in [34].

One key issue in a monitoring scheme is the reliable and robust identification of damage in the considered system. Among numerous procedures that have been developed over the last decades, there is one vibration-based approach that has already been successfully applied to different industrial structures such as bridges and offshore structures [75]. This method which is based on the normalized cumulative spectral energy of vibration response measurements has also been applied to identify a fatigue crack in an industrial pipe system during laboratory tests. It is expected that this technique can also be applied to certain components in natural gas pipelines such as compression stations. For an application to buried and underwater transmission pipes, adaptations depending on the sensor technique used would be necessary.

Information about the structural integrity based on monitoring is one component of a framework that was developed to estimate certain risks linked with industrial processes as input into decision processes [2]. This framework has been implemented and applied, for instance, to estimate the risk of failure due to extreme wind and wave conditions of an offshore wind turbine structure.

Reliable information about the risk of structural failure is one of several factors that need to be taken into account if decisions with respect to maintenance, rehabilitation, strengthening or renewal of industrial facilities have to be

made. In case of systems such as natural gas pipelines not only technical and financial but also environmental and societal criteria have to be included into the decision process. Risk management of natural gas pipelines is therefore a multi-objective task involving multiple criteria.

This gave reason to apply multicriteria approaches such as the elimination and choice expressing reality approach (ELECTRE) [12] and multi-attribute utility theory (MAUT) [11], [1], [40] to the risk assessment of gas pipelines. These algorithms were designed to assign alternative solutions of a problem into categories such as 'acceptable', 'unacceptable' or 'intermediate' [18]. An overview about different multicriteria and multi-objective decision making techniques are given by [17] and [46].

5 Conclusions

As natural gas pipelines consist essential lifelines of modern industrial societies, their potential damage (i.e., from deterioration to not only results into an interruption of supply but it can also have severe and sometimes irreversible consequences.

Therefore, resilience and robustness of natural gas pipelines has attracted scientific attention as a means to guarantee safety as well as disruptive and reliable operation and service. In this context, monitoring and inspection of pipelines are important elements to reliably assess their structural integrity.

This paper reviews, in a comparative sense where possible, numerous state-of-the-art and practice techniques and methods for inspection and monitoring of natural gas pipelines. Successful cases studies are also reported along with respective limitations and challenges, the latter including installation and optimal sensor placement issues. The paper concludes with future developments are also discussed in light of risk assessment procedures and life-cycle management of pipeline networks.

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