Eleventh U.S. National Conference on Earthquake Engineering
Integrating Science, Engineering & Policy
June 25-29, 2018
Los Angeles, California

SAFETY FACTORS OF BURIED STEEL NATURAL GAS PIPELINES UNDER SPATIALLY VARIABLE EARTHQUAKE GROUND MOTION

N. Psyrras[1], A. Sextos[2], O.-S. Kwon[3] and S. Gerasimidis[4]

ABSTRACT

The damaging potential of spatial variability in seismic ground motion on the integrity of buried pipelines is demonstrated in this paper. A numerical analysis methodology is developed first to determine the seismic demand of a typical straight steel natural gas pipeline running through a site composed of two different media with an impedance ratio of 2 and swept by vertically propagating SV-waves. The analysis follows a sub-structured, two-phase approach involving the computation of pipeline input excitation from 2D linear viscoelastic and linear-equivalent seismic site response models and the quasi-static application of the derived critical motion profiles on a near-surface 3D continuum soil model surrounding an extended inelastic shell model of the pipeline. The focus is then placed on identifying the ground and exciting conditions bearing adverse effects on the peak pipeline response. By comparing the pipeline demand in terms of stress and strain to capacity metrics prescribed in present seismic codes, the importance of the local site response is gauged. Results show that low-frequency ground vibrations produce the most unfavorable demand on the pipe for the set of cases examined. More importantly, even though pipeline axial strain demand-to-capacity ratios for elastic local site response under weak excitation imply a large safety margin, pipeline demand can exceed capacity near the site boundary under strong excitations and subsequent nonlinear soil response. Plastic local buckling may also develop in the pipeline under high-intensity input motions, thus highlighting the necessity to account for non-synchronous earthquake ground motion in case of horizontally nonhomogeneous sites.

[1] PhD researcher, Dept. of Civil Engineering, University of Bristol, UK (e-mail: n.psyrras@bristol.ac.uk)
[2]Assoc. Professor, Dept. of Civil Engineering, University of Bristol, UK
[3]Assoc. Professor, Dept. of Civil Engineering, University of Toronto, Canada
[4]Ass. Professor, Dept. of Civil Engineering, University of Massachusetts at Amherst, USA

Psyrras N., Sextos A., Kwon O.-S., Gerasimidis S. On the safety factors of buried steel natural gas pipelines under spatially variable earthquake ground motion. *Proceedings of the 11th National Conference in Earthquake Engineering*, Earthquake Engineering Research Institute, Los Angeles, CA. 2018.

Eleventh U.S. National Conference on Earthquake Engineering
Integrating Science, Engineering & Policy
June 25-29, 2018
Los Angeles, California

Safety factors of buried steel natural gas pipelines under spatially variable earthquake ground motion

N. Psyrras[1], A. Sextos[2], O.-S. Kwon[3], S. Gerasimidis[4]

ABSTRACT

The damaging potential of spatial variability in seismic ground motion on the integrity of buried pipelines is demonstrated in this paper. A numerical analysis methodology is developed first to determine the seismic demand of a typical straight steel natural gas pipeline running through a site composed of two different media with an impedance ratio of 2 and swept by vertically propagating SV-waves. The analysis follows a sub-structured, two-phase approach involving the computation of pipeline input excitation from 2D linear viscoelastic and linear-equivalent seismic site response models and the quasi-static application of the derived critical motion profiles on a near-surface 3D continuum soil model surrounding an extended inelastic shell model of the pipeline. The focus is then placed on identifying the ground and exciting conditions bearing adverse effects on the peak pipeline response. By comparing the pipeline demand in terms of stress and strain to capacity metrics prescribed in present seismic codes, the importance of the local site response is gauged. Results show that low-frequency ground vibrations produce the most unfavorable demand on the pipe for the set of cases examined. More importantly, even though pipeline axial strain demand-to-capacity ratios for elastic local site response under weak excitation imply a large safety margin, pipeline demand can exceed capacity near the site boundary under strong excitations and subsequent nonlinear soil response. Plastic local buckling may also develop in the pipeline under high-intensity input motions, thus highlighting the necessity to account for non-synchronous earthquake ground motion in case of horizontally nonhomogeneous sites.

Introduction

The ever-growing need for environmentally friendly energy sources has established natural gas (NG) as a norm in the energy market. It becomes hence apparent that the associated infrastructure must be protected against natural and man-made hazards. Onshore long-distance transportation of natural gas is typically performed by large-diameter, high-pressure, shallow-buried steel pipelines (transmission system). Although the design of such structures is not earthquake-driven, they still

[1] PhD researcher, Dept. of Civil Engineering, University of Bristol, UK (e-mail: n.psyrras@bristol.ac.uk)
[2] Assoc. Professor, Dept. of Civil Engineering, University of Bristol, UK
[3] Assoc. Professor, Dept. of Civil Engineering, University of Toronto, Canada
[4] Ass. Professor, Dept. of Civil Engineering, University of Massachusetts at Amherst, USA

Psyrras N., Sextos A., Kwon O.-S., Gerasimidis S. On the safety factors of buried steel natural gas pipelines under spatially variable earthquake ground motion. *Proceedings of the 11th National Conference in Earthquake Engineering*, Earthquake Engineering Research Institute, Los Angeles, CA. 2018.

have to meet certain code-specified performance criteria. When crossing earthquake-prone areas, the transmission lines may be exposed to significant seismic risk, thus leading to several cases of reported damage [1–5]. An issue that has not yet been investigated in adequate depth is that, due to their extended length, NG pipelines may be subjected to ground motions that considerably differ spatially in terms of their amplitude, phase and frequency content. This is especially true when seismic waves propagate through ground with non-uniform properties, giving rise to local site effects. Local site response results from material stiffness gradients (i.e., inhomogeneity), topographic features at the surface (e.g., hills and canyons) or special subsurface morphologic conditions, such as sediment-filled valleys and cavities. The combination of any of these local features with a suitably oriented oncoming seismic wavefield may cause ground motion amplification, modify the frequency content and induce normal ground strains and curvatures (e.g., [6–9]), which are known to govern the earthquake behavior of buried pipelines [10–12].

The study of the effects of spatially varying earthquake ground motion (SVEGM) on the seismic behavior of buried pipelines has been driven by different considerations, most notably regarding the representation of the pipeline as a beam on elastic foundation or shell, the properties of the surrounding soil (i.e., homogeneous or nonhomogeneous) and the inclusion or not of contact effects at the soil-pipe interface. Solutions appeared early in the literature for the peak elastic ground strains in homogeneous soil due to wave arrival delay [13,14], which, by extension, apply also to the pipeline if interaction is ignored. Sakurai and Takanashi [15] observed experimentally that limited axial soil-pipe interaction occurs for low-intensity excitations, while Shinozuka and Koike [16] consolidated analytically that pipeline inertia is negligible. Hindy and Novak [17] developed a matrix formulation of the full dynamic problem for elastic pipelines under P- or S-waves both in homogeneous sites and in sites made up of two discrete media. It was found that the peak axial and bending stresses due to body waves propagating parallel to the pipeline occur near the boundary of the two media, and are larger than the ones in the homogeneous case. Wong *et al.* [18,19] derived analytical descriptions of the coupled soil-pipeline motion under various wavefronts using the theory of elastodynamics along with shell equations for the pipeline. Ignoring soil-pipe interaction, Kouretzis *et al.* [20] presented elastic solutions for the axial and hoop strain fields of long cylindrical structures due to travelling harmonic waves in elastic halfspace and uniform soil over bedrock. Furthermore, stochastic analysis has been employed to quantify the effect of seismic motion incoherence on the pipeline response [21,22].

A loading scenario briefly described in Hindy and Novak [17] is examined herein: a pipeline crossing at right angles the interface between two vertically adjacent soil strata with distinctly different shear wave velocities. The seismic excitation is assumed to be in the form of vertically propagating plane S-waves, resulting principally in alternating axial compression-extension of the pipeline. The problem is solved numerically by the finite element method using non-trivial techniques and large-scale modelling of a long soil domain. The objective is to quantify the influence of the linear and nonlinear local ground response of this particular site on the predicted seismic demand of the pipeline and to compare the demand with relevant performance limits available in the latest seismic and pipeline-specific standards.

Generation of the pipeline excitation

To estimate the pipeline stresses and deformations, the complete dynamic soil-pipeline interaction problem is treated in two phases in order to reduce some of the involved complexities. First, the time-variant local ground response to SV-wave patterns is obtained by performing 2D seismic site response analysis. One major limitation in almost of the previous wave propagation studies, that is, the linearity of the soil, is lifted in this study, as both linear viscoelastic and linear-equivalent soil models are analyzed here. From the computed response histories at soil points at the level of the pipeline (assumed to be buried at a depth of 1m from the crown), the critical spatial profile of displacements is extracted corresponding to the maximum and minimum axial strain. Because the seismic response of buried pipelines is not inertia-governed, the dynamic terms in the coupled governing equations of motion can be dropped and a quasi-static analysis may be deemed a reasonable approximation. If we further invoke the observation that inertial and kinematic interaction between the pipeline and the soil is in most cases insignificant [12,16,23], the soil displacements corresponding to the critical timeframe can be applied as static boundary conditions on a near-field soil model in contact with a pipeline model after due consideration of the sliding and separation potential between pipe and soil. An outline of this approach is given in Fig. 1.

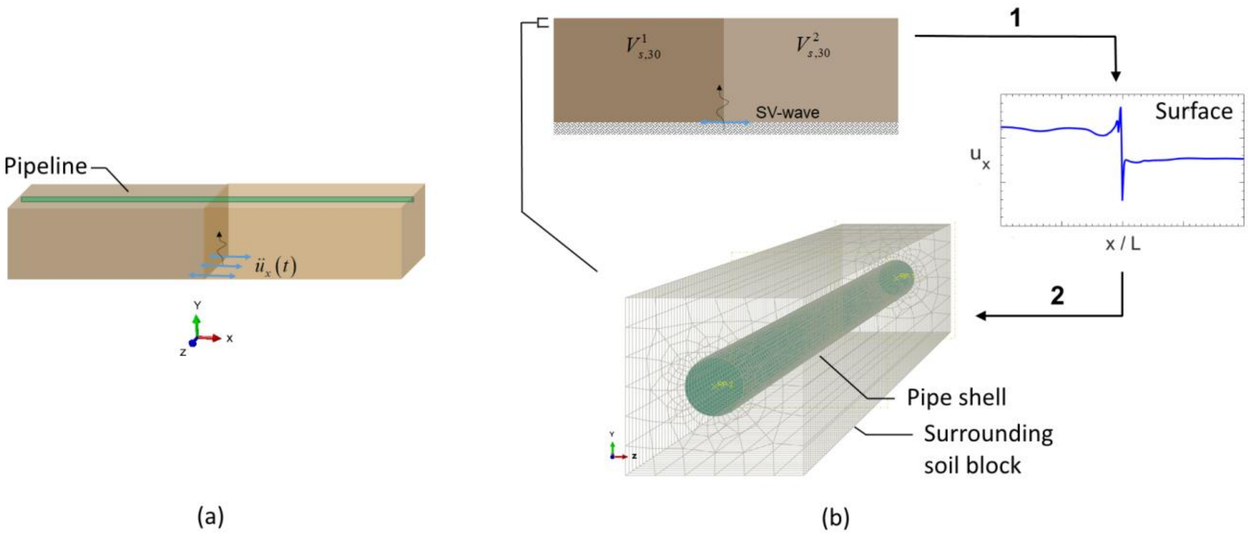

Figure 1. (a) The problem at hand; (b) flowchart of the proposed methodology: (1) 2D seismic site response analysis and extraction of the critical differential displacement profile of the soil; (2) application of the critical soil displacement profile on the near-field soil-pipeline system and evaluation of the pipeline demand.

Finite element model of the soil

A composite 30m-deep sandy site resting on elastic rock is considered with low-strain shear modulus G_o increasing with depth as described by the empirical formula of Eq. 1 [24]:

$$G_o=1000\cdot K_{2,max}(\sigma'_m)^{0.5} \tag{1}$$

where σ'_m is the average effective confinement stress and $K_{2,max}$ a constant with values that can be found in the literature [25] as a function of the relative density D_r. A dense (D_r = 90%) and loose (D_r = 30%) deposit are assumed in contact, with mass densities ρ_1=2 Mg/m^3 and ρ_2=1.4 Mg/m^3 and average S-wave velocities of 280 and 200 m/s, respectively, giving an impedance contrast of

2. Fig. 2a illustrates the physical problem and identifies its basic parameters. The generated discrete *S*-wave velocity profiles are plotted in Fig. 3a. The nonlinearity of the soil under strong input motions is accounted for in an approximate way using 2D equivalent-linear soil models, where the element-specific shear moduli and critical damping ratios are independently adjusted at the end of each linear dynamic analysis to match shear strain-compatible values. For this iterative scheme, the strain measure used is the maximum shear strain. The dynamic soil properties are represented by the analytical Darendeli curve set [26]. Because this model is pressure-dependent, a different *G-D-γ* curve is assigned to sublayers partitioned at every 10m depth, based on an averaged effective confinement stress (Fig. 3b). The initial critical damping ratio is taken ξ_{in}=2% and the Poisson ratio is 0.33. Damping is of the Rayleigh type.

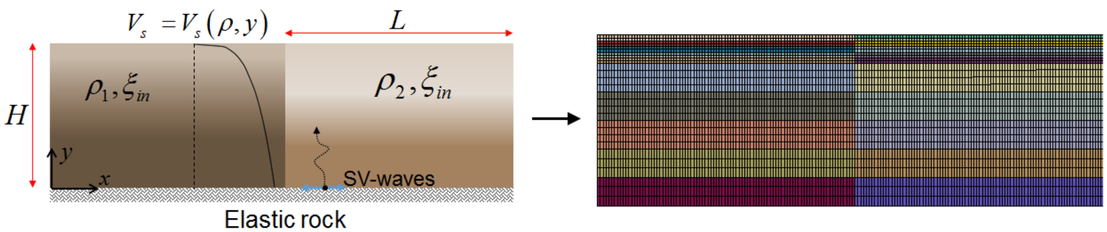

Figure 2. Identification of the significant parameters of the mathematical idealization of the problem (left) and spatial discretization and layering of the domain (right).

The domain discretization in the vertical sense was carried out in a way to ensure adequate resolution for exciting frequencies of up to 15 Hz in the range of wave velocities examined in this study (Fig 2b). The horizontally unbounded medium was rendered finite for the purposes of the FE analysis by truncating it at a distance of L=150m (half-length) from both sides of the vertical boundary. This choice was based on a convergence study of the induced axial ground strain distributions. Standard viscous boundaries [27] were placed at the base of the deposits to absorb the scattered seismic waves, while horizontal rollers were assigned to the lateral boundaries. Typical values were assumed for the bedrock properties, namely V_s=1000 m/sec and ρ=2.4 Mg/m^3.

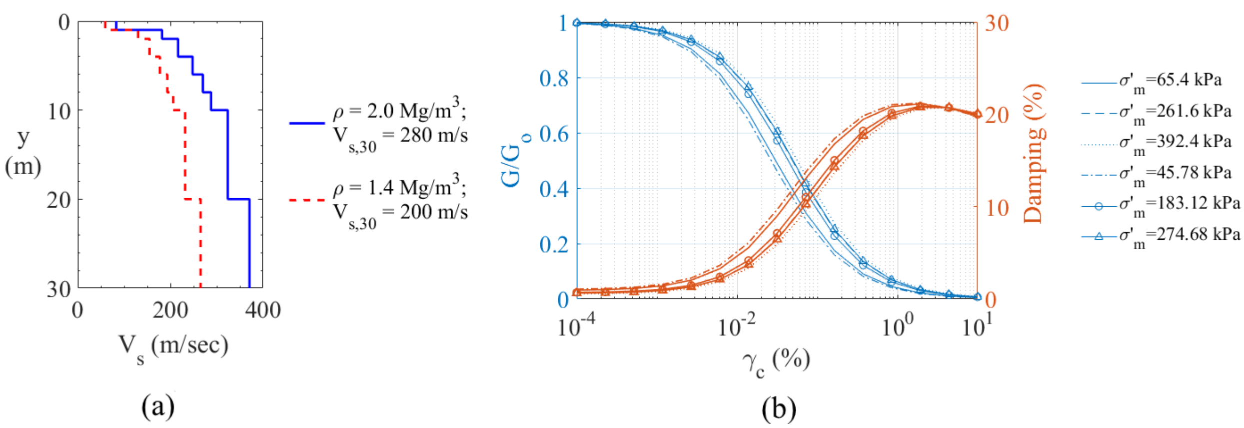

Figure 3. (a) The generated shear wave velocity profiles for the two deposits of the site under consideration; (b) calculated shear moduli degradation and damping curves.

To investigate the effect of the frequency content of the excitation on the ground deformation, Ricker wavelets with different predominant frequencies f_p of 0.5, 1 and 5 Hz were used to represent input acceleration pulses at the bedrock. Excitations as weak as 0.05g were assumed at the bedrock propagating vertically. Bedrock motions were further scaled to 0.2g and 0.3g to trigger nonlinear soil response. Their waveforms for bedrock peak ground acceleration *PGA*=0.2g are shown in Fig. 4. Site response analysis was performed using the computer code QUAD4M [28].

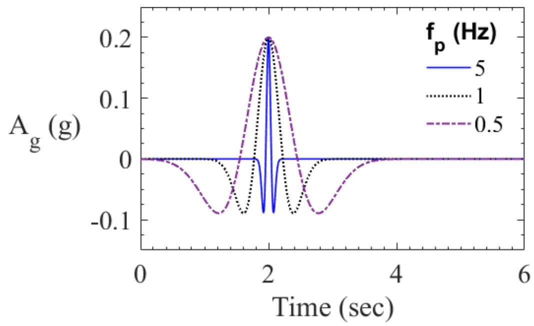

Figure 4. Acceleration time-series of Ricker wavelets with varying f_p and amplitude 0.2g.

Verification

The finite element modeling was verified against closed-form 1D wave propagation solutions computed by an alternative computer code, DeepSoil [29]. A uniform, damped soil on elastic rock was considered with equivalent-linear properties described by the Seed and Idriss curves [24] (mean limit). The Friuli earthquake record (Tolmezzo station, 1976) was applied as a sample horizontal base excitation. The obtained transfer functions for the 2D and 1D representations are plotted in Fig. 5, providing a satisfactory match. Minor differences are mainly attributed to the different damping formulations used by the two codes in terms of frequency dependence.

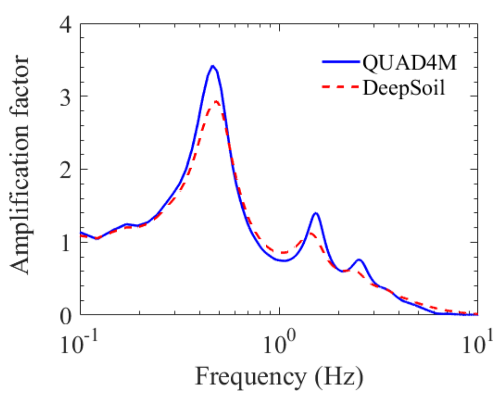

Figure 5. Comparison of surface-to-bedrock motion amplification factors between 1D and 2D equivalent-linear site response models.

Critical deformation profiles

Due to length limitations, only results for strong excitation of 0.3g amplitude are presented here. The ground axial strain, horizontal displacement, axial curvature and vertical displacement are plotted in Fig. 5 against the normalized horizontal coordinate. The maximum compressive strains are marked as well. It is seen that the peak deformation occurs for the 1 Hz pulse, causing a sharp strain peak with magnitude 12.2% and a marked ‘parasitic’ downward movement of 0.14m. Of note, the results are quite sensitive to the type and number of *G-D-γ* curves employed in the model.

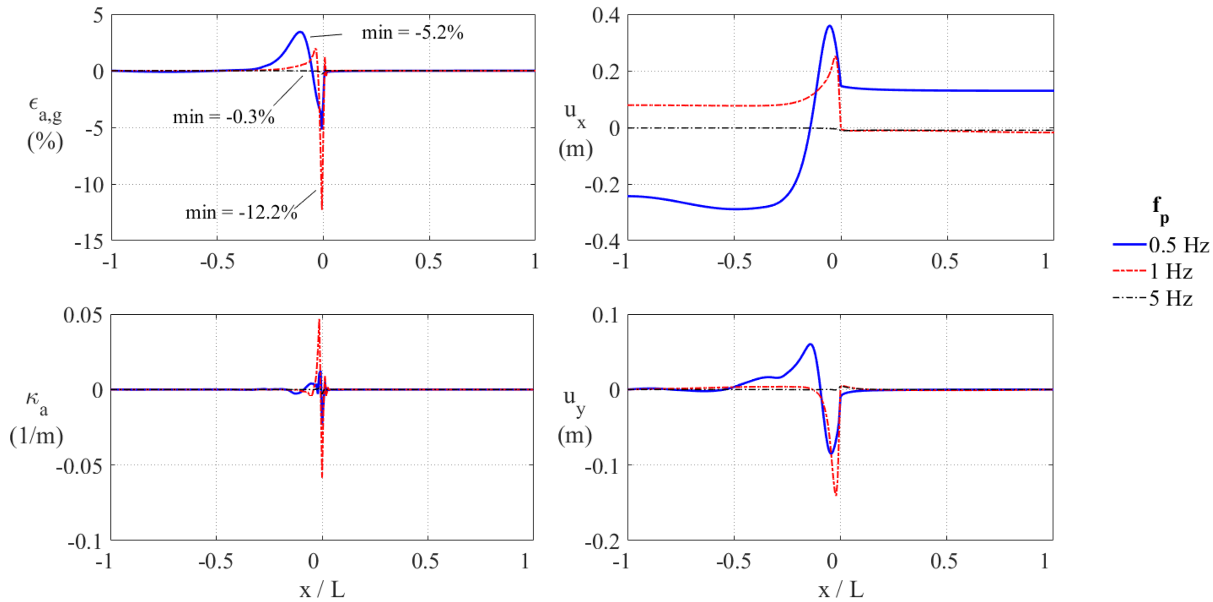

Figure 6. Critical spatial profiles of ground response quantities parallel to the pipeline axis for acceleration Ricker pulses of 0.3g amplitude.

Soil-pipeline interaction analysis

In phase 2, the critical linear and nonlinear in-plane soil deformation in terms of axial strains and curvatures obtained from phase 1 for f_p=1 Hz are prescribed as an external load on the 300m-long soil-pipe system. Next, an incremental static nonlinear analysis is carried out in Abaqus. Notably, the imposed displacements only vary in the horizontal dimension; they are invariant with depth because the depth of the simulated domain is small compared to the longest wavelength of the impinging waves examined ($3D/\lambda_{max}$<0.10), therefore the in-plane motion of the soil particles does not change much in this dimension.

Finite element model of the soil-pipe system

We chose to model the pipeline as a shell and the near-field soil as a 3D continuum to capture the actual gravity- and pressure-induced stress field in the pipeline. In this manner, we also tackled the uncertainty of using soil-pipe springs to model the soil restraint. A pipeline with R=500mm, t=12mm, σ_y=450 MPa, σ_y/σ_{ult}=0.85, ε_{ult}=4% and P/P_y=0.63, where P_y is the yield pressure, is considered. The cross-section of the system is sketched in Fig. 7a, where the dashed lines show the truncation of the half-plane. Plastic behavior is considered for steel through classical flow plasticity and a von Mises criterion. The uniaxial tensile behaviour is described by a Ramberg-Osgood fit to an elastoplastic curve with isotropic hardening. The soil medium is assumed linear elastic based on the converged G at the end of iterations as presented in phase 1.

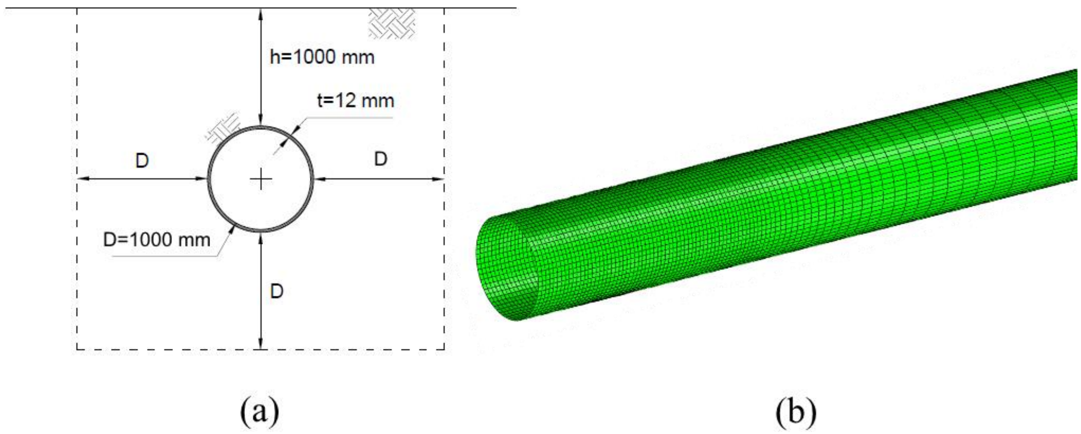

Figure 7. (a) The truncated cross-section of the soil-pipeline system used in the 3D model; (b) view of the pipeline mesh with progressive refinement towards the site boundary.

The interface frictional behavior is dictated by a Coulomb friction law to allow sliding, while separation between the soil and the pipe walls is possible. The normal and tangential interface behaviors are coupled. Assuming an angle of frictional resistance equal to 30° for the soil, an average interface friction coefficient of 0.4 is used [12]. Boundary conditions for the pipeline shell are assumed completely free at its end sections. Four-node Abaqus S4R elements are used for the pipeline and eight-node C3D8R for the soil. The element discretization of the pipeline is shown in Fig. 7b.

Results

The response of the pipeline is presented in the form of demand-to-capacity ratios (equivalent to the reciprocal of factor of safety) along a straight line of points. The following strain and stress limits are used to define pipeline capacity:

- the allowable compressive strain ε_c under wave propagation as defined by:
  - Eurocode 8 (EC8) [30]: min{1, 20 t/R} (%)
  - American Lifeline Alliance (ALA) [31]: 0.75[0.25 t/R–0.0025+3000$(PR/Et)^2$] (%)
  - Japan Gas Association (JGA) [32]: 3%
- the allowable axial stress σ_{xx} as specified by ASME [33]: $0.9\sigma_y$
- the Mises yield criterion: $\sigma_{VM} < \sigma_y$

Compression limits were chosen over tension ones because they are more conservative. Results are shown in Fig. 8. For the low intensity pulse, the axial strain demand in the mid-section was found, as anticipated, negligible (0.005%), with the pipeline responding in the elastic range. The increased strains close to the pipe ends arise artificially as a result of the pressure-induced Poisson deformation effect and the loosely restrained pipe ends. The appreciable level of stress developed in the pipeline is also a mere result of the internal pressure. On the contrary, for the high intensity pulse, the peak produced axial strain occurs in the plastic range and is 2 and 1.5 times larger than the conservative non-buckling limits of EC8 and ALA, respectively. In terms of stresses, the maximum von Mises value was found to be 97% of the ultimate strength. The ASME axial stress limit is satisfied, but it is the strain demand that is more of interest in this type of buried infrastructure. As a last remark not illustrated here, at a certain point after yield, wrinkling appears in the perfect pipe and gradually localized bulges start to form, denoting shell buckling failure.

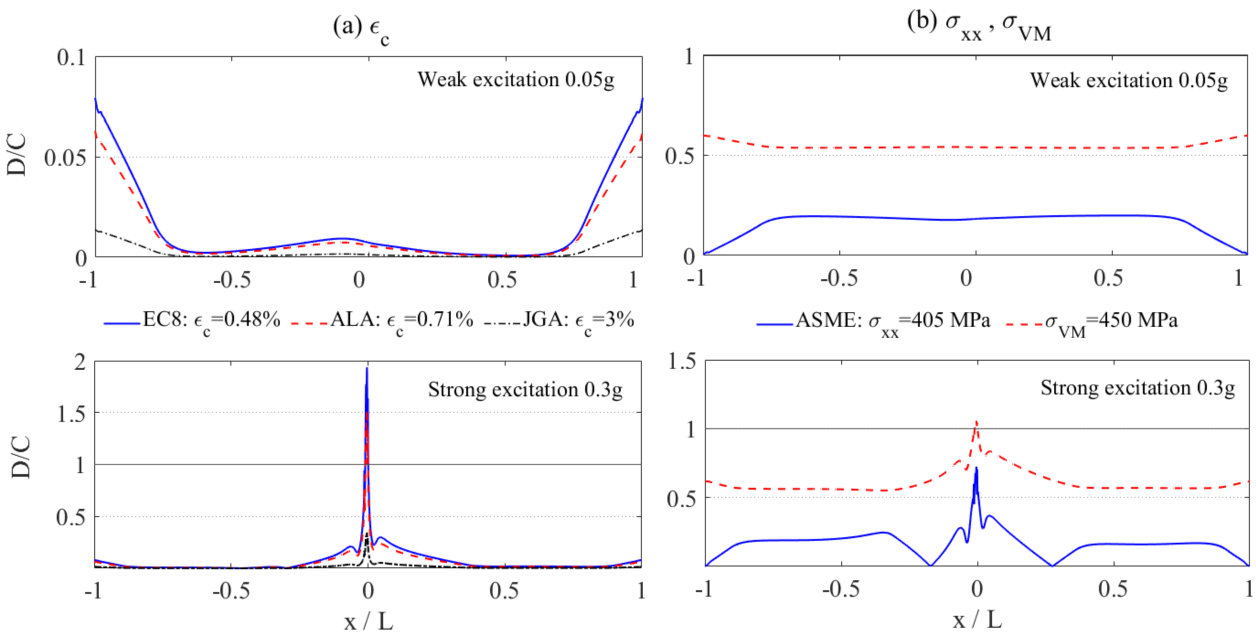

Figure 8. Pipeline demand-to-capacity ratios along the crown line in terms of (a) compressive axial strains and (b) stresses, for 0.05g (top) and 0.3g (bottom), 1 Hz Ricker acceleration pulse.

Conclusions

A numerical analysis methodology is introduced in this paper with the aim to assess the seismic performance of transmission natural gas steel pipelines buried in nonhomogeneous sites under high-intensity excitations, considering the strong soil nonlinearity of local ground response. The scenario examined involves a straight pipeline laid through a dry cohesionless site consisting of two deposits of different stiffness, with their interface perpendicular to the pipe section, and excited by pulse-like SV-waves. To the degree that the dynamic response of the pipeline and the inertial-kinematic interactions are negligible, the most important findings of this study are summarized below:

- Out of phase vibrations of the adjacent soil deposits generate axial normal strains and curvatures, the magnitude of which depends heavily on the exciting frequency. Their dependence on other factors, such as depth to bedrock and 1D/2D soil structure, remains to be investigated.
- Pipeline axial strain demand for elastic local site response under weak excitation is found to be at least two orders of magnitude lower than the code-specified capacities. The presence of considerable stresses is attributed to the in-situ state of the pipeline.
- When the soil profiles respond asynchronously under stronger earthquake intensity and within the nonlinear range, the peak demand exceeds capacity even for conservative code limits.
- In the worst-case scenario examined, the pipeline starts manifesting deformation localization in the vicinity of the ground strain peak, which is an early indication of instability collapse.

Overall, this study, even though by no means exhaustive, demonstrates that conventionally designed NG transmission pipelines crossing soft sites with varying stiffness may experience remarkably high axial distress under strong seismic input motions, with demand measures exceeding by a large margin the limits established in the current state-of-practice. This calls for further investigation of the critical combination of conditions that can lead to ultimate limit states such as local buckling and reconsideration of the existing allowances.

Acknowledgements

The authors would like to acknowledge the financial support of the Horizon 2020 Programme of the European Commission through grant MSCA-RISE-2015-691213-EXCHANGE-Risk (www.exchange-risk.eu). The first author would also like to express his gratitude to the UK Engineering and Physical Sciences Research Council for sponsoring his doctoral studies.

References

1. Housner GW, Jenningst PC. The san fernando california earthquake 1972; **1**(August 1971): 5–31. DOI: 10.1002/eqe.4290010103.
2. Hall JF, Holmes WT, Somers P, Institute EER. *Northridge earthquake of January 17, 1994: reconnaissance report*. Earthquake Engineering Research Institute; 1996.
3. EQE Summary Report. *The January 17, 1995 Kobe Earthquake*. 1995.
4. O'Rourke M. Wave Propagation Damage to Continuous Pipe. *Tclee 2009* 2009: 1–9. DOI: 10.1061/41050(357)76.
5. Esposito S, Giovinazzi S, Elefante L, Iervolino I. Performance of the L ' Aquila (central Italy) gas distribution network in the 2009 (M w 6 . 3) earthquake 2013; **2009**: 2447–2466. DOI: 10.1007/s10518-013-9478-8.
6. Zhang B, Papageorgiou AS. Simulation of the response of the marina district Basin, San Francisco, California, to the 1989 Loma Prieta earthquake. *Bulletin of the Seismological Society of America* 1996; **86**(5): 1382–1400.
7. Assimaki D, Kausel E, Gazetas G. Wave propagation and soil-structure interaction on a cliff crest during the 1999 Athens Earthquake. *Soil Dynamics and Earthquake Engineering* 2005; **25**(7–10): 513–527. DOI: 10.1016/j.soildyn.2004.11.031.
8. Gelagoti F, Kourkoulis R, Anastasopoulos I, Tazoh T, Gazetas G. Seismic wave propagation in a very soft alluvial valley: Sensitivity to ground-motion details and soil nonlinearity, and generation of a parasitic vertical component. *Bulletin of the Seismological Society of America* 2010; **100**(6): 3035–3054. DOI: 10.1785/0120100002.
9. Scandella L, Paolucci R. Earthquake induced ground strains in the presence of strong lateral soil heterogeneities. *Bulletin of Earthquake Engineering* 2010; **8**(6): 1527–1546. DOI: 10.1007/s10518-010-9186-6.
10. Kubo K. Fundamental Concept Of Aseismic Design Of Underground Piping Systems. *Proc. 5th European Conf. Earthq. Engng*, Istanbul, Turkey: 1975.
11. Shinozuka M, Koike T. *Estimation of Structural Strains in Underground Lifeline Pipes*. 1979.
12. O'Rourke MJ, Hmadi K El. Analysis of continuous buried pipelines for seismic wave effects. *Earthquake Engineering & Structural Dynamics* 1988; **16**(6): 917–929. DOI: 10.1002/eqe.4290160611.
13. Newmark NM. Problems in wave propagation in soil and rock. *Proceedings of the international symposium on wave propagation and dynamic propertiesof earth materials*, Albuquerque: University of New Mexico Press; 1968.

14. St John CM, Zahrah TF. Aseismic design of underground structures. *Tunnelling and Underground Space Technology* 1987; **2**(2): 165–197. DOI: 10.1016/0886-7798(87)90011-3.

15. Sakurai A, Takanashi T. Dynamic Stresses of Underground Pipelines During Earthquakes. *Proceed. of 4th World Conf. on Earthq. Engng.*, Santiago, Chile: 1969.

16. Shinozuka M, Koike T. ESTIMATION OF STRUCTURAL STRAINS IN UNDERGROUND LIFELINE PIPES. 1979: 31–48.

17. Hindy A, Novak M. Earthquake response of underground pipelines. *Earthquake Engineering & Structural Dymamics* 1979; **7**: 451–476.

18. Wong K, Datta S, Shah A. Three-Dimensional Motion of Buried Pipeline. I: Analysis. *Journal of Engineering Mechanics* 1986; **112**(12): 1319–1337. DOI: 10.1061/(ASCE)0733-9399(1986)112:12(1319).

19. Wong BKC, Shah AH, Asce M. THREE-DIMENSIONAL MOTION OF BURIED PIPELINE . II : NUMERICAL RESULTS Numerical results are presented for a model simulating a concrete shell buried in a soil-like or rock-like material . The material properties of the pipe and the host materials in the ha 1987; **112**(12): 1338–1345.

20. Kouretzis GP, Bouckovalas GD, Gantes CJ. 3-D shell analysis of cylindrical underground structures under seismic shear (S) wave action. *Soil Dynamics and Earthquake Engineering* 2006; **26**(10): 909–921. DOI: 10.1016/j.soildyn.2006.02.002.

21. Zerva A, Ang A. S, Wen YK. A Study of Seismic Ground Motion for Lifeline Response Analysis. *Civil Engineering Studies, Structural Research Series (University of Illinois at Urbana-Champaign, Department of Civil Engineering)* 1985(521).

22. Zerva A. On the spatial variation of seismic ground motions and its effects on lifelines. *Engineering Structures* 1994; **16**(7): 534–546. DOI: 10.1016/0141-0296(94)90089-2.

23. Kubo K. Behavior of Underground Water pipes During an Earthquake. *Proc. 5th World Conf. Earthquake Eng.*, Rome: 1974.

24. Seed HB, Idriss IM. *Soil moduli and damping factors for dynamic response analyses*. Berkeley, Calif. : College of Engineering, University of California; 1970.

25. Kramer SL. *Geotechnical earthquake engineering*. Upper Saddle River.: Upper Saddle River. : Prentice Hall; 1996.

26. Darendeli MB. Development of a new family of normalized modulus reduction and material damping curves 2001.

27. J. L, R. K. Finite dynamic model for infinite media. *J Eng Mech Div* 1969; **95**(EM4): 859–877.

28. Hudson M, Idriss I, Beikae M. QUAD4M: a computer program to evaluate the seismic response of soil structures using finite element procedures and incorporating a compliant base. *Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, CA* 1994(January).

29. Hashash YMA, Musgrove MI, Harmon JA, Groholski DR, Phillips CA, Park D. DEEPSOIL 6.1, User Manual 2016.

30. European Committee for Standardization (CEN). Eurocode 8: Design of structures for earthquake resistance—Part 4: Silos,tanks and pipelines (EN 1998-4: 2006). *European Committee for Normalization, Brussels* 2003; **2**(2005).

31. American Lifeline Alliance. *Guideline for the Design of Buried Steel Pipe*. vol. 2001. 2005.

32. Japan Gas Association. Seismic Design for Gas Pipelines 2000: 91–100.

33. American Society of Mechanical Engineers. ASME B31.4 Pipeline Transportation Systems for Liquids and Slurries 2012; **2002**.