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Seismic fragility of buried steel natural gas pipelines due to axial compression at geotechnical discontinuities

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Abstract: This paper presents an extended set of numerical fragility functions for the structural assessment of buried steel natural gas (NG) pipelines subjected to axial compression caused by transient seismic ground deformations. The study focuses on NG pipelines crossing sites with a vertical geotechnical discontinuity, where high compression straining of a buried pipeline is expected to occur under seismic transient ground deformations. A de-coupled numerical framework is developed for this purpose, which includes a 3D finite element model of the pipe-trench system employed to evaluate rigorously the soil-pipe interaction effects on the pipeline axial response in a quasi-static manner. One-dimensional soil response analyses are used to determine critical ground deformation patterns at the vicinity of the geotechnical discontinuity, caused by the ground shaking. A comprehensive parametric analysis is performed by implementing the proposed analytical framework for an ensemble of 40 recorded earthquake ground motions. Crucial parameters that affect the seismic response and therefore the seismic vulnerability of buried steel NG pipelines namely, the diameter, wall thickness, burial depth and internal pressure of the pipeline, the backfill compaction level, the pipe-soil interface friction characteristics, the soil deposits characteristics, as well as initial geometric imperfections of the walls of the pipeline, are systematically considered. The analytical fragility functions are developed in terms of peak ground velocity (PGV) at the ground surface, for four performance limit states, considering all the associated uncertainties. The study contributes towards a reliable quantitative risk assessment of buried steel NG pipelines, crossing similar sites, subjected to seismically-induced transient ground deformations.

Keywords: *natural gas pipelines; seismic response; fragility curves; soil-pipe interaction; transient ground deformations; steel pipelines; local buckling*

1. Introduction

Earthquake-induced damage on Natural Gas (NG) pipeline networks may lead to significant downtimes, which in turn may result in high direct and indirect economic losses. The 1999 Chi-Chi earthquake in Taiwan, for instance, caused noticeable damage on natural gas supply systems, with the associated economic loss for the major natural gas companies exceeding \$25

1 million (Chen et al. 2000; Lee et al. 2009) More importantly, severe damages may trigger
2 ignition or explosions with life-threatening consequences and significant effects on the
3 environment. The 1995 Hyogo-Ken Nambu earthquake in Japan is a rather devastating
4 example since the particular earthquake caused gas leakages from buried pipelines at 234
5 different locations, which subsequently led to more than 530 fires (EQE 1995; Scawthorn et al.
6 1995). The above aspects highlight the importance of simple, yet efficient, methods for
7 structural vulnerability assessment of NG pipeline networks.

8 Buried steel NG pipelines were found to be quite vulnerable to high strain imposed by
9 permanent ground deformations, associated with seismically-induced ground failures, i.e. fault
10 movements, landslides, liquefaction-induced settlements or uplifting and lateral spreading
11 (O'Rourke M.J. and Liu 1999). Although to a lesser extent, transient ground deformations,
12 induced by seismic wave propagation, have also contributed to seismic damage of steel
13 pipelines (Housner and Jennings 1972; O'Rourke T.D. and Palmer 1994; O'Rourke M.J.
14 2009). Indeed, transient ground deformation may trigger diverse damage modes on continuous
15 NG pipelines, including (i) shell-mode or local buckling, (ii) beam-mode buckling, (iii) pure
16 tensile rupture, (iv) flexural bending failure and (v) excessive ovaling deformation of the
17 section (O'Rourke M.J. and Liu 1999). Recent studies have demonstrated that pipelines
18 embedded in heterogeneous sites and/or subjected to asynchronous ground seismic motions are
19 likely to be further affected by appreciable deformations and strains due to transient ground
20 deformations, which in turn may lead to buckling damages on the pipeline (Psyrras and Sextos
21 2018; Psyrras et al. 2019).

22 In practice, the seismic risk assessment of pipelines is mainly performed, by implementing
23 empirical fragility relations, which were constructed on the basis of observations of the
24 behaviour of buried pipelines during past earthquakes (e.g. ALA 2001; NIBS 2004). A detailed
25 review of available empirical relations may be found in Tsiniidis et al. (2019a). These relations
26 normally provide correlations between the pipeline *repair rate*, RR , i.e. the number of pipe
27 repairs per unit of pipeline length, and a selected seismic *intensity measure*, expressing the
28 seismic intensity.

29 The majority of available fragility relations refer to cast-iron or asbestos cement segmented
30 pipelines, the seismic response of which is quite distinct compared to continuous pipelines,
31 such as buried NG pipelines (O'Rourke M.J. and Liu 1999). The lack of relevant damage
32 reports and therefore of relevant fragility relations for continuous pipelines has been attributed
33 by some researchers to their better performance, compared to the segmental pipelines, when
34 subjected to seismically-induced transient ground deformations. However, as stated above,
35 under certain conditions, transient ground deformations may impose significant strains on
36 buried pipelines.

37 The implementation of *repair rate* as an engineering demand parameter (EDP) does not
38 provide any information regarding the severity of damage, as well as the type of required
39 repair, while the accuracy of the repair reports that constitute the basis for the development of
40 empirical fragility functions may be debatable, since these are commonly drafted after a short
41 period from the main event and under the pressure for rapid restorations. Moreover, available
42 empirical relations were developed on the basis of damage reports on pipeline networks found

1 in USA and Japan, whilst in southern Europe or other seismic prone areas there is an evident
2 lack of relevant information. Evidently, the applicability of the empirical fragility relations is
3 restricted to cases where the network characteristics, e.g. pipe dimensions and materials, soil
4 conditions etc, and the ground motion characteristics, are similar to the relevant characteristics
5 of the sample used to develop the relations. Finally, available fragility relations do not
6 disaggregate between distinct damage modes and associated effects on the structural integrity
7 and serviceability of the pipeline. Along these lines, a general and unconditional use of these
8 relations might introduce a degree of uncertainty in the seismic risk assessment of networks
9 with distinct characteristics (Psyrras and Sextos 2018).

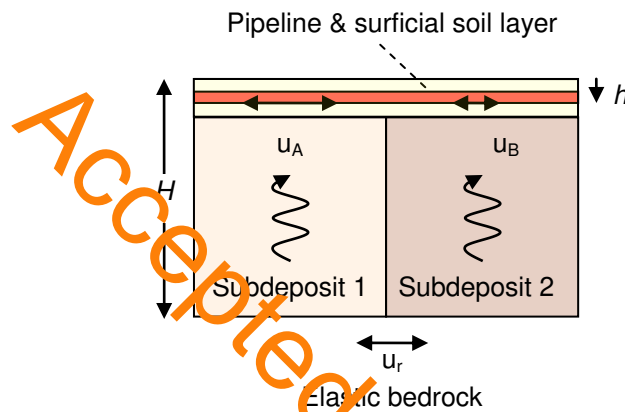
10 A limited number of numerical fragility curves that compute probabilities of failure for well-
11 defined limit states in the ‘classical sense’ have been proposed, recently (Lee et al. 2016;
12 Jahangiri and Shakid 2018). However, available numerical fragility functions refer to rather
13 limited number soil-pipe configurations and do not cover NG pipelines with diameters larger
14 than 800 mm that are commonly used in transmission NG networks. More importantly, the
15 relevant numerical studies do not examine thoroughly salient parameters that may affect the
16 response and hence the vulnerability of buried NG pipelines under seismically-induced
17 transient ground deformations, such as the effects of the internal operational pressure of the
18 pipeline or the initial geometric imperfection of the walls of the pipes and the spatial variability
19 of soil conditions.

20 In the light of the above considerations and knowledge gaps, this paper presents an extended
21 set of numerical fragility curves for the structural assessment of buried steel NG pipelines
22 subjected to axial compression caused by transient seismic ground deformations. The study
23 focuses on pipelines crossing perpendicularly a vertical geotechnical discontinuity with an
24 abrupt change on the soil properties, where the potential of high compression strain and
25 therefore buckling failures is expected to be increased under seismic transient ground
26 deformations. A detailed analytical framework is developed for this purpose, which is
27 employed in a comprehensive parametric analysis of large number of pipe-soil configurations
28 and for an ensemble of 40 recorded earthquake ground motions. Crucial parameters affecting
29 the response of buried steel pipelines namely the diameter, wall thickness, burial depth and
30 internal pressure of the pipeline, the existence of initial geometric wall imperfections of the
31 pipeline, the trench soil compaction level, the pipe-backfill interface friction characteristics and
32 the variability of the characteristics of the soil deposits, are thoroughly accounted for in the
33 study. The analytical fragility curves are developed in terms of peak ground velocity (*PGV*) at
34 the ground surface, for four performance limit states, considering the associated uncertainties.

36 2. Definition of problem

37 Fig. 1 illustrates schematically the problem examined herein. A continuous buried steel NG
38 pipeline of external diameter D and wall thickness t is embedded in a surficial block of soil at a
39 burial depth h . The surficial block of soil is resting over a soil deposit with a vertical
40 geotechnical discontinuity. The latter divides the deposit into two subdeposits, i.e. subdeposit 1
41 and subdeposit 2, with abrupt changes on their physical and mechanical properties. The total
42 depth of the soil deposit is H . It is noted that the selected soil deposits constitute idealized

1 cases to facilitate the numerical parametric study presented herein. The soil-pipe system is
 2 subjected to ground seismic shaking, in the form of upward propagated, vertically polarized
 3 plane shear waves, which causes a dissimilar ground movement of the adjusted subdeposits.
 4 The dissimilar ground movement of the adjusted soil subdeposits produces a differential
 5 horizontal ground deformation along the pipeline axis near the critical section of the
 6 geotechnical discontinuity, which subsequently is transferred via the pipe-trench soil interface
 7 on the pipeline causing its compressive-tensional axial straining. A potential high axial
 8 compression strain of the pipeline might finally lead to a failure of the pipeline in the form of
 9 local buckling. Based on the above considerations, a numerical framework is developed to
 10 evaluate the vulnerability of the embedded steel NG pipeline under an ensemble of carefully
 11 selected real records.



12
 13
 14 **Fig. 1** Schematic view of the examined problem (H : depth of the soil deposit, h : burial depth of the
 15 pipeline, u_r : seismic displacement at bedrock, u_A and u_B : horizontal seismic deformations of the adjacent
 16 soil subdeposits).

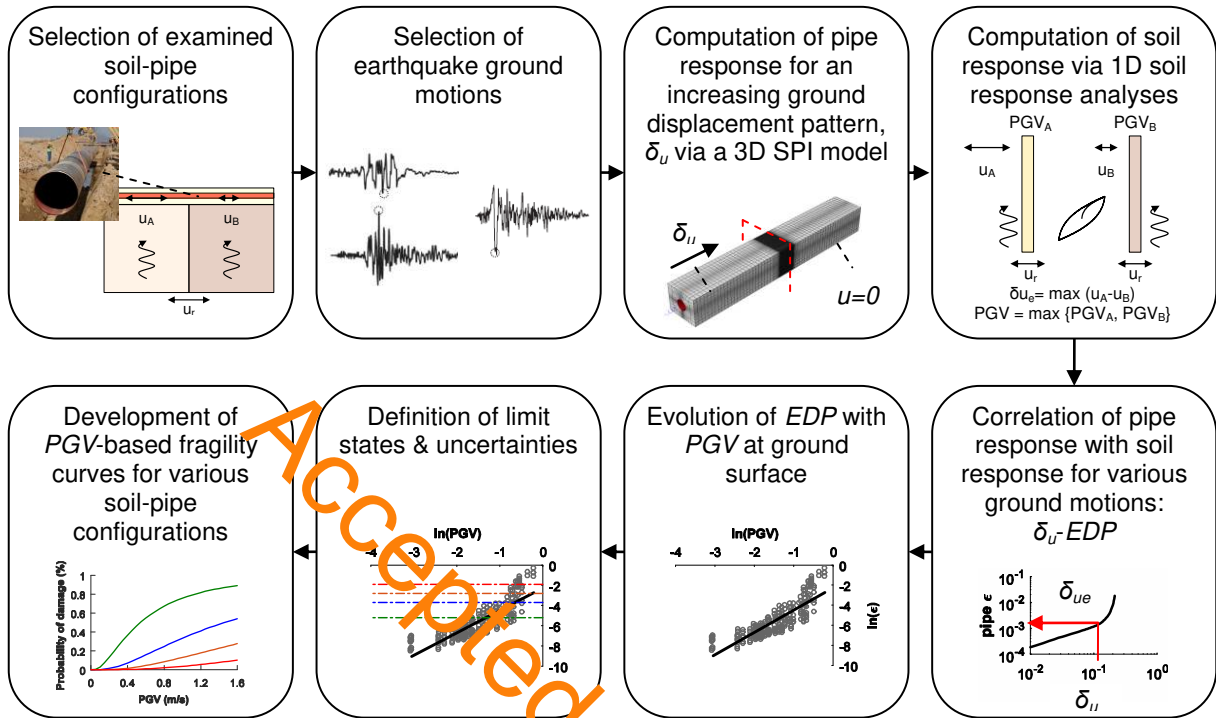
18 3. Analytical framework

19 3.1 General flowchart

20 The extended dimensions of the problem in hand, the need for refined meshes to capture
 21 potential buckling failure of the pipeline, the complexity in simulating the material and
 22 geometrical nonlinearities, i.e. sliding and/or detachment phenomena at the soil-pipe interface,
 23 during ground shaking, the uncertainty in the definition of the characteristics of heterogeneous
 24 soil sites, and issues associated with proper selection of seismic ground motions, are all reasons
 25 that render a fully 3D time history analysis of the coupled pipeline-trench soil system
 26 computationally prohibitive (Psyrras and Sextos 2018).

27 Generally, the inertial soil-structure interaction (SSI) effects are not considered to be a key
 28 factor in the context of the dynamic soil-pipe interaction (SPI) problem mainly due to the
 29 reduced mass of the pipe in comparison to that of the soil (O'Rourke M.J. and Hmadi 1988).
 30 This allows for a decoupling of the problem in successive stages in order to reduce the high
 31 computational cost, associated with a fully-fledged 3D SPI dynamic analysis. Moreover, it
 32 allows for the investigation of the effect of transient ground deformation on the response of the
 33 embedded pipeline in a quasi-static manner.

1 Based on the above considerations, an analytical framework was developed within this study to
 2 evaluate thoroughly the seismic vulnerability of NG pipelines embedded in sites, similar to Fig.
 3 1. The flowchart of the analytical framework is illustrated schematically in Fig. 2.



5
6
7 **Fig. 2** Flowchart of the analytical framework for the development of fragility curves for buried NG
8 pipelines crossing sites with a geotechnical discontinuity.

9
10 The analysis of the seismic response of the selected soil-pipe configurations is carried out in
11 steps, as follows: initially, a 3D trench-pipe numerical model is developed to compute the axial
12 compression response of the buried steel NG pipeline under an increasing level of seismically-
13 induced relative axial ground displacement, δ_u , considering the soil compliance effects. The
14 response of selected soil deposits is then computed by means of separate 1D nonlinear soil
15 response analyses of the adjacent subdeposits. In particular, through the soil response analyses,
16 the horizontal deformations of the subdeposits are computed for the selected ground motions at
17 the burial depth of the examined pipelines. These are subsequently used to define maximum
18 differential ground movement patterns, δ_{ue} , of the soil deposits consisting of the examined
19 subdeposits. The soil response analyses are also used to calculate the peak ground velocity
20 PGV at the ground surface, which is used as seismic intensity measure (IM) to express the
21 fragility curves. The outcomes of the 3D SPI analyses and 1D soil response analyses are finally
22 combined, to correlate the pipe response, in terms of maximum axial compression strain, ϵ ,
23 which is selected as engineering demand parameter, EDP , for the pipeline, with the ground
24 response computed for each of the selected pairs of subdeposits and each ground motion. The
25 latter combinations result in relationships of pipe strain ϵ with the PGV at ground surface,
26 which are finally used to define fragility curves for four predefined performance limit states,

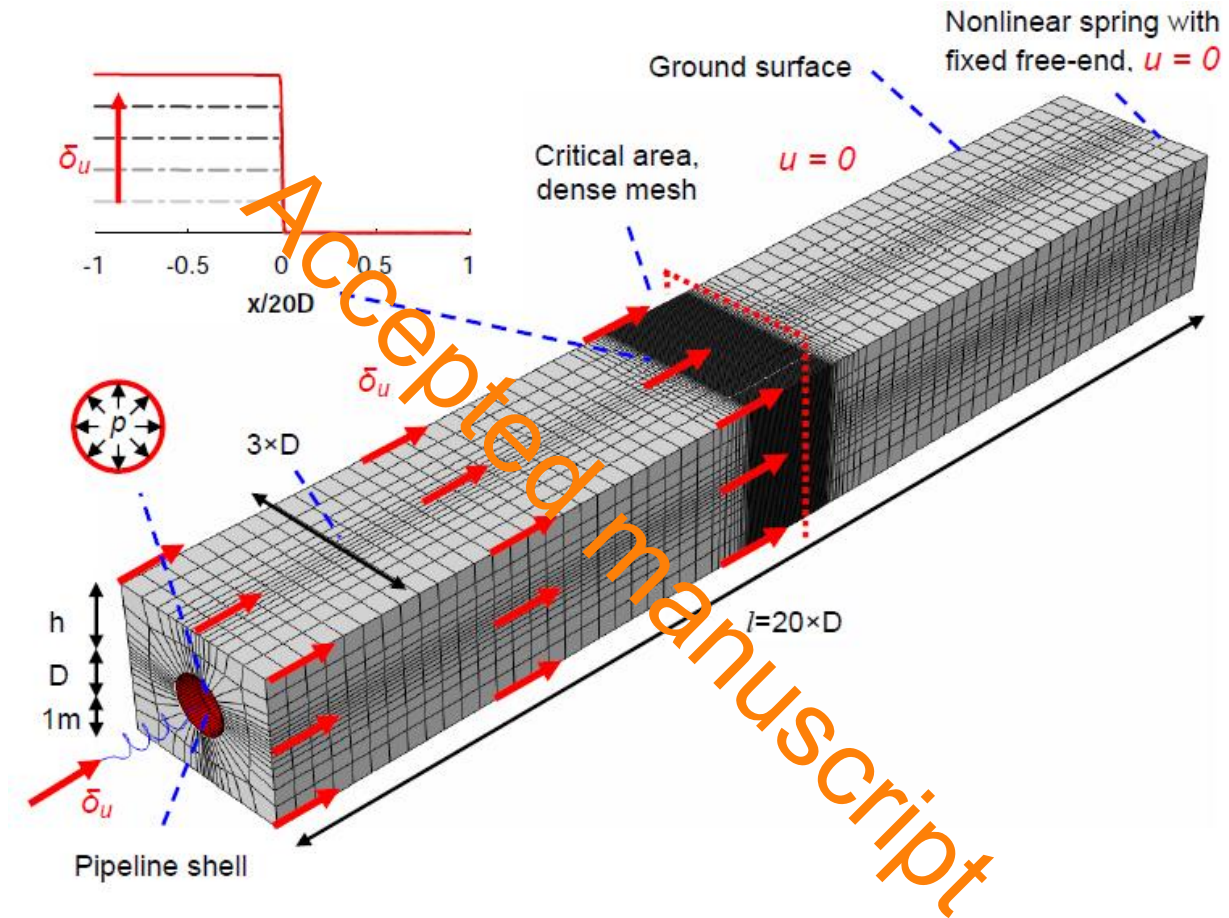
1 considering all the associated uncertainties. The analytical framework is further elaborated in
2 the following sections.

3

4 **3.2 3D trench-pipe model for SPI analysis**

5 A 3D model of the trench soil, encasing a cylindrical shell model of the pipeline, is initially
6 developed in [ABAQUS \(2012\)](#), aiming at computing the axial response of the pipeline under
7 an increasing level of horizontal relative ground displacement, δ_u , developed near a
8 geotechnical discontinuity ([Fig. 3](#)).

9



10

11

12 **Fig. 3** 3D trench-pipe numerical model for the computation of the axial compression response of the
13 pipeline, under an increasing level of seismically-induced relative ground displacement, δ_u .

14

15 The use of the near field 3D continuum trench-pipe model allows for a rigorous simulation of
16 localized buckling modes that might potentially be developed in the pipe under axial
17 compression, as well as for the proper simulation of geometric imperfections of the pipeline
18 walls, which are expected to affect significantly the axial compression response of the buried
19 pipeline ([Kyriakides et al. 1990](#); [Tsinidis et al. 2018](#); [Psyrras et al. 2019](#)). Additionally, it
20 allows for a proper simulation of the operational pressure of the pipeline and contact nonlinear
21 phenomena, i.e. sliding and/or potential detachment in the normal direction, between the
22 pipeline wall and the surrounding ground. The latter is of great importance since the shear
23 behaviour of the trench soil-pipe interface effectively controls the level of shear stresses that

1 are transmitted along the perimeter of the pipeline during shaking. The integral of these shear
 2 stresses constitutes the axial loading of the pipeline.

3.2.1 Dimensions of the 3D model

5 The shallow burial depth of the pipeline in addition to **absence of significant inertial SSI effects**
 6 and the assumption of in-plane ground deformation pattern, allow for the simulation of only the
 7 surficial soil-trench, which constitutes a surficial block from the semi-infinite 3D ground
 8 domain (Psyrras et al. 2019). Along these lines, the distance between the side boundaries of the
 9 trench model and the pipe edges is set equal to one pipe diameter, whereas the distance
 10 between the pipe invert and the bottom boundary of the trench model is set equal to 1.0 m.
 11 Evidently, the distance between the pipe crown and ground surface is defined on the basis of
 12 the adopted burial depth, h , of the examined pipeline.

13 Generally, an ‘adequately long’ 3D continuum model is required to replicate the actual SPI
 14 phenomena, accounting for the ‘anchorage’ length of the pipeline on the surrounding trench
 15 and its effect on the transmitted shear stresses from the trench on the pipeline through the soil-
 16 pipe interface during the axial deformation of the trench. Additionally, there is a requirement of
 17 fine discretization of the pipe to adequately resolve the buckling modes of the pipeline, as
 18 discussed in the following. The above aspects increase significantly the relevant computational
 19 cost of the analysis, even if the seismic loading is considered in a quasi-static manner. To
 20 reduce the required length of the 3D model, while accounting for the effect of the infinite
 21 pipeline length on the response of the examined pipeline-trench soil configuration, nonlinear
 22 springs are introduced at both sides of the pipeline. The force-displacement relation of the
 23 nonlinear springs, which are act parallel to the pipeline axis, is given by the following
 24 relations:

$$F = \begin{cases} \lambda EA \delta_x & \text{for } \delta_x \leq \frac{\tau_{\max}}{k_s} \\ \lambda EA \frac{\tau_{\max}}{k_s} + \frac{\pi D \tau_{\max}}{m} \left(\sqrt{\left(\lambda \frac{\tau_{\max}}{k_s} \right)^2 + 2m \left(\delta_x - \frac{\tau_{\max}}{k_s} \right)} - \left(\lambda \frac{\tau_{\max}}{k_s} \right) \right) & \text{for } \delta_x > \frac{\tau_{\max}}{k_s} \end{cases} \quad (1)$$

26 where:

$$\lambda = \sqrt{\frac{\pi D k_s}{EA}} \quad (2)$$

$$m = \frac{\pi D \tau_{\max}}{EA} \quad (3)$$

29 δ_x is the ground-pipe relative axial movement caused by the relative axial ground deformation
 30 δ_u as a result of the dissimilar ground movement of the adjacent subdeposits, τ_{\max} is the
 31 maximum shear resistance that develops along the trench backfill-pipe interface, k_s is the shear
 32 stiffness of the trench backfill-pipe interface and EA is the axial stiffness of the pipeline cross
 33 section. For cohesionless backfills, the maximum shear resistance depends on the adopted
 34 friction coefficient μ and varies along the perimeter of the pipe. Average values of τ_{\max} and k_s

1 may be computed on the basis of numerical simulations of simple axial pull-out tests of the
 2 examined pipe from the examined trench soil, as discussed later. Following the above
 3 simulation approach, the required length of the 3D pipe-soil trench model may be reduced to
 4 20 times the external diameter of the pipeline. The theoretical background behind this
 5 simulation approach, which is inspired by a numerical model that was developed by [Vazouras
 et al. \(2015\)](#), is presented in [Tsinidis et al. \(2019b\)](#). The validity of this proposed simulation is
 7 verified in the following, by comparing the stresses and strains computed at the middle critical
 8 section of a selected pipeline by the 3D reduced-length model with the nonlinear springs, with
 9 relevant predictions of an equivalent quite extended, almost ‘infinitely’ long 3D continuum
 10 model of the soil-pipe configuration subjected to the same axial ground deformation pattern.
 11 Typical static boundary conditions are applied at the bounding soil surfaces, while the ground
 12 surface is set free.

3.2.2 Finite element discretization

15 The trench soil is simulated by means of hexahedral (brick-type) elements with equivalent soil
 16 properties being assigned to them (i.e. soil degraded stiffness), the latter being estimated by the
 17 separate 1D soil response analyses, as discussed in the ensuing. Inelastic, reduced integration
 18 *S4R* shell elements are used to mesh the pipeline. The particular shell elements have both
 19 membrane and bending stiffness. The mesh density of the pipeline at the central section of the
 20 3D model, i.e. at the assumed location of the geotechnical discontinuity, where the axial strain
 21 of the pipeline is expected to maximize under ground shaking, is selected to be fine enough, so
 22 that to resolve the inelastic buckling modes of an equivalent axially compressed unconstrained
 23 cylindrical steel shell ([Psyrras et al. 2019](#)). To facilitate the selection of mesh, the half-
 24 wavelength in the post-elastic range, $\lambda_{c,p}$, is initially computed for the selected pipelines, as
 25 per [Timoshenko \(1961\)](#):

$$\lambda_{c,p} \approx \lambda_{c,el} \times \sqrt{E_p/E} \quad (4)$$

27 where E is the Young’s modulus of the steel grade of the pipeline, E_p is the plastic modulus of
 28 the steel grade of the pipeline and $\lambda_{c,el}$ the elastic axial half-wavelength. Considering a
 29 Poisson’s ratio $\nu = 0.3$ for the steel grades examined herein, the latter is given as:

$$\lambda_{c,el} \approx 1.72\sqrt{Rt} \quad (5)$$

31 where R is the radius of the pipeline and t is the wall thickness of the pipeline. Assuming that
 32 the plastic modulus E_p is equal to $0.1E$, [Eq. 4](#) yields: $\lambda_{c,p} \approx 0.5\lambda_{c,e}$ ([Psyrras et al., 2019](#)).
 33 Element lengths, ranging between 1.0 cm and 2.0 cm, depending on the geometric properties of
 34 the selected pipelines, were found capable to reproduce the theoretical axial half-wavelength
 35 $\lambda_{c,p}$ of the examined pipelines. These mesh seeds are applied over a length of 2.0 m in the
 36 middle section of the examined pipelines. The mesh density away from the critical central zone
 37 is gradually decreased, with the axial dimension of the shell elements being as high as 0.30 m,
 38 in an effort to reduce the computation cost. The mesh discretization of the trench soil in the
 39 axial direction of the model matches the exact mesh seed of the pipeline to avoid any initial

1 gaps during the generation of mesh. The mesh seed of the trench in the other two directions is
2 restricted to 0.30 m.

3.2.3 Trench soil-pipe interface

5 The soil-pipe interface is simulated by means of an advanced hard contact interaction model,
6 available in [ABAQUS \(2012\)](#). The model allows for sliding and/or potential detachment in the
7 normal direction between the interacting pipe and trench-soil elements during the horizontal
8 deformation of the surrounding trench. The shear behaviour of the interface model is controlled
9 by the classical Coulomb friction model, through the introduction of a friction coefficient, μ .
10 The adopted values of friction coefficients, μ , are presented in *Section 4*.

3.2.4 Behaviour of the backfill soil and pipe

13 The surficial soil-backfill is simulated as an elastic medium, with equivalent properties (i.e.
14 degraded soil stiffness) defined as per *Section 3.3*. The plastic behaviour of the steel pipelines
15 is modelled employing a classical flow plasticity model combined with a von Mises yield
16 criterion. The model is defined by fitting Ramberg-Osgood curves ([Equation 6](#)) to bilinear
17 isotropic curves, the latter describing the tensile uniaxial behaviour of the selected steel grades
18 (see *Section 4*)

$$\varepsilon = \frac{\sigma}{E} + a \left(\frac{\sigma}{\sigma_y} \right)^n \quad (6)$$

20 where E is the elastic modulus, σ is the axial stress, ε is the axial strain, σ_y is the yield stress,
21 n is a hardening parameter and a is a 'yield offset' which is equal to $a \sigma_y / E$. Parameters a and
22 n are defined for the selected steel grades of the examined pipelines in *Section 4*.

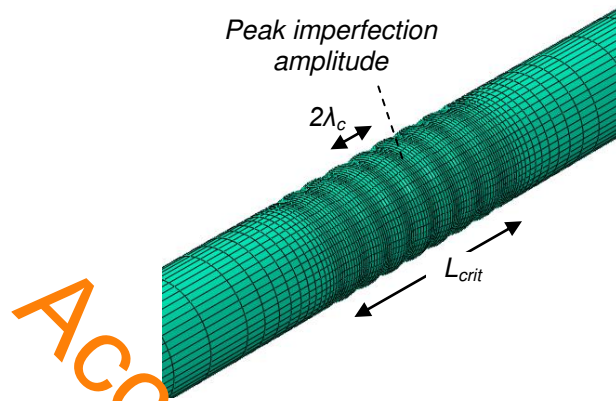
3.2.5 Initial geometric imperfection of the pipeline section

25 The axial compressive response of thin-walled steel pipelines is known to be highly affected by
26 initial geometric imperfections of the walls ([NASA 1968](#); [Yun and Kyriakides 1990](#); [Psyrras et al. 2019](#)). To account for this effect on the structural response of the examined pipelines, both
27 'perfect' pipelines and equivalent pipelines with initial geometric imperfections are examined.
28 For the latter cases, a stress-free, biased axisymmetric imperfection is considered. The
29 imperfection is defined on the basis of a sinusoid modulated by a second sinusoid, as per
30 [Equation \(7\)](#), following [Psyrras et al. \(2019\)](#):

$$\bar{w}(x) = \left[w_0 + w_1 \cos \left(\frac{\pi x}{N \lambda_c} \right) \right] \cos \left(\frac{\pi x}{\lambda_c} \right), \quad -\frac{L_{crit}}{2} \leq x \leq \frac{L_{crit}}{2}, \quad L_{crit} = 2.0m, \quad 2N \lambda_c = L_{crit} \quad (7)$$

33 The positive values correspond to outward direction from the mid-surface of the pipeline shell
34 wall. The peak amplitude of the imperfection is set as a function of the pipe lining thickness,
35 based on the following formulation: $w = w_0 + w_1 = 0.10t$. The imperfection level is based on
36 relevant specifications from NG pipeline manufactures, e.g., ArcelorMittal specifies a
37 manufacturing tolerance for the walls of API-5L X65 pipelines in the range of + 15% to -
38 12.5% ([ArcelorMittal 2018](#)). The imperfection is applied over the central critical pipeline zone

1 with length equal to $L_{crit} = 2.0$ m, centered at the exact position, where the soil discontinuity is
 2 considered. Fig. 4 illustrates a detail of the mesh of the central section of an imperfect pipeline.
 3 The exact same perturbation is introduced on the mesh of the trench soil, surrounding the
 4 pipeline, to prevent any initial gaps during the generation of the mesh that might affect the
 5 contact phenomena during loading. It is noted that any residual stresses on the pipelines,
 6 related to the manufacturing process were disregarded in the present study.
 7



8
 9 **Fig. 4** Detail of the mesh of the central section of a 914.4 mm pipeline with a biased axisymmetric
 10 geometrical imperfection of the walls (the radial deformation is exaggerated by a scale factor, i.e. $\times 10$).
 11

12 3.2.6 Analysis steps

13 The stress state, associated with the gravity and the internal pressure of the pipeline, is initially
 14 established within a general static step. The effect of seismically-induced transient ground
 15 deformation is then introduced in quasi-static fashion. In particular, the nodes of the one half of
 16 the trench model and the free node of the relevant nonlinear spring are fixed in the axial
 17 direction, i.e. the right-hand side of the model in Fig. 3, whereas the nodes of the other half of
 18 the trench model and the free node of the relevant nonlinear spring are monotonically forced to
 19 move towards constraint part of the model, in a stepwise fashion, thus resulting in a relative
 20 axial displacement of the trench model equal to δ_u , the latter increasing throughout the analysis
 21 step. The analysis is carried out till the numerical analysis collapses, i.e. after buckling failure
 22 of the examined pipeline. This displacement configuration is equivalent to the case, where both
 23 halves of the trench model, are moving dissimilarly in the axial direction, causing the same
 24 differential axial ground movement δ_u on the examined system. The displacement pattern is
 25 kept constant with depth coordinates over the trench soil domain and the free-ends of the
 26 nonlinear springs, for the sake of simplicity. This assumption is considered valid since the
 27 depth of the trench domain is rather small compared to the predominant wavelength of
 28 common seismic waves (Psyrras et al. 2019). The above kinematic loading induces shear
 29 stresses along the pipe-soil interface, which in addition to the axial loading induced on the both
 30 ends of the pipeline through the nonlinear springs, result in an axial compression straining of
 31 the pipeline. The latter is traced for the increasing level of relative axial ground displacement,
 32 δ_u , via a modified Riks solution algorithm, available in ABAQUS (ABAQUS 2012). Through
 33 this analysis, a curve describing the relation between an increasing relative axial ground
 34 displacement, δ_u , and the corresponding peak compression axial strain ϵ of the critical middle

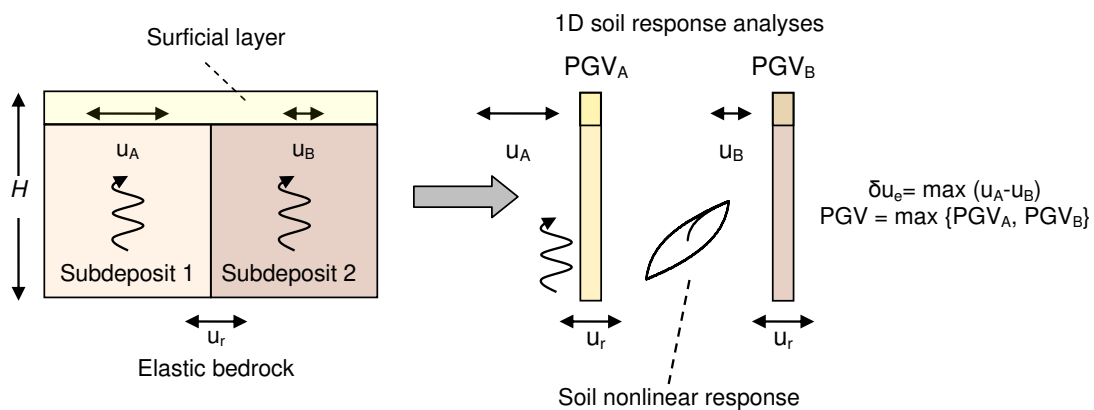
1 section of the pipeline, i.e. near the assumed geotechnical discontinuity, can be established.
 2 The peak compression axial strain is evaluated as the envelope of the compression axial strains
 3 computed for all the shell elements that are located within the critical section of the pipeline. It
 4 should be highlighted that the analysis focuses on the axial ground displacements and
 5 disregards the vertical ones that might be observed near geotechnical discontinuities since the
 6 former constitute the dominant loading mechanism for the buried pipeline. Since the response
 7 of the pipeline is computed for an increasing level of relative axial ground displacement, δ_u , the
 8 outcome of one 3D SPI analysis may be used to examine the axial straining of the pipe under a
 9 variety of selected ground axial relative displacements, δ_{ue} , caused by diverse seismic motions.
 10 This is of course possible under the assumption and implementation of mean equivalent soil
 11 properties for the trench backfill soil, corresponding to the strain-range that is anticipated for
 12 the selected ground seismic motions.

13

14 3.3 1D soil response analysis of adjacent soil deposits

15 The response of the selected sites is evaluated through separate 1D soil nonlinear response
 16 analyses of the adjacent subdeposits (Fig. 5), carried out by employing the code DEEPSOIL
 17 v6.1 (Hashash et al. 2016). The hysteretic nonlinear response of the soil during ground shaking
 18 is considered in the analyses by means $G-\gamma-D$ curves, which are properly selected for the
 19 examined deposits, following Darendeli (2001). To avoid the potential amplification of higher
 20 frequencies of the ground that may result in unrealistic oscillations of the acceleration time
 21 histories in low ground strains, additional viscous damping of 1 % is also introduced in the
 22 form of the frequency-dependent Rayleigh type (Hashash and Park 2002). The Rayleigh
 23 coefficients are properly tuned for a frequency interval range, characterizing the ‘dominant
 24 frequencies’ of each soil column. Through the soil response analyses, time histories of the
 25 horizontal deformations of the soil columns are calculated at the burial depths of the pipelines.
 26 These time histories are subsequently used to compute maximum differential ground
 27 deformation patterns δ_{ue} for the selected pairs of adjusted subdeposits. Additionally, time
 28 histories of the horizontal velocity are computed at the ground surface, which are used to
 29 evaluate the peak ground velocity PGV at ground surface. The latter is used as seismic intensity
 30 measure for the development of the analytical fragility curves.

31



32

33 **Fig. 5** Schematic view of the analysis framework used to compute the response of the selected soil sites
 34 under ground shaking.

3.4 Correlation of 3D SPI and 1D soil response analyses results

The critical relative axial ground deformation patterns, δ_{ue} , which are defined on the basis of the results of the 1D soil response analyses, are correlated with the predicted straining of the pipeline, using the δ_u -maximum compressive axial strain ε relations computed by the 3D SPI analyses. The identified from the above correlation procedure pipeline strains ε are employed to define ε -PGV relationships, which are finally used to define the numerical fragility curves for predefined limit states in a probabilistic framework, accounting for all the associated uncertainties.

3.5 Performance limit states

The development of analytical fragility curves requires the rigorous definition of performance limit states, which are associated with specific damage levels. In this study, four performance limit states are defined on the basis of peak axial compressive strain ε of the pipeline, following [Jahangiri and Shakir \(2018\)](#). The limit states, summarized in [Table 1](#), refer to different return periods of earthquake, ranging between 25 and 2475 years, and are associated with different levels of damage on the pipeline. They are actually defined on the basis of thorough review of relevant studies, guidelines, codes and regulations, as per [Table 1](#). Despite this fact, the definition of limit states contains a level of uncertainty, which is considered in the definition of fragility curves, as discussed in the ensuing. The first two limit states, i.e. operable limit state (OLS) and pressure integrity limit state (PILS), may be characterized as operational limit states, since no leakages are expected, and the flow of the pipeline is not disrupted. On the contrary, ultimate limit state (ULS) and global collapse limit state (GCLS), constitute ultimate limit states, since pipe wall tearing is expected, resulting in leakages and flow disruption. In terms of damage level, the four limit states may be associated with slight, moderate, extensive and complete damage, respectively. For a more detailed presentation of the relevant definitions of the limit states the reader is referred to [Jahangiri and Shakir \(2018\)](#).

3.6 Development of fragility functions

Fragility functions describe the probability of exceeding different performance limit states, given a level of ground shaking intensity. Following common approaches, the fragility relations in this study are developed, by employing a lognormal probability distribution function:

$$P(ds \geq ds|S) = \Phi \left[\frac{1}{\beta_{tot}} \times \ln \left(\frac{PGV}{PGV_{mi}} \right) \right] \quad (8)$$

where $P(ds \geq ds|S)$ is the probability of exceeding a particular limit state, ds , for a given seismic intensity level, the latter defined by the peak ground velocity, PGV , at ground surface. Φ is the standard cumulative probability function, PGV_{mi} is the median threshold value of PGV , required to cause the i^{th} damage state, and β_{tot} is the total lognormal standard deviation. Based on the above definitions, the analytical fragility curves may be sufficiently described by defining the following parameters PGV_{mi} and β_{tot} .

1 **Table 1.** Limit states adopted [Jahangiri and Shakib \(2018\)](#), t : thickness of the pipeline, D : diameter of
 2 the pipeline

Limit state	Maximum limit axial compressive strain	Description	Return period (years)	ε_c defined following:
Operable Limit State (OLS)	$\varepsilon = \min(0.01, 0.4 \times t/D)$	Despite some minor plastic deformations, the pipeline will operate immediately after the event.	25	ALA (2001); JG(G)-206-03 (2004); EN 1998-4 (2006)
Pressure Integrity Limit State (PILS)	$\varepsilon = \min(0.04, 1.76 \times t/D)$	Despite some significant deformations on the pipe, no leakage of containment is taken place.	95	ALA (2001); JG(G)-206-03 (2004); CEN (2006); Mohareb (1995); Honegger et al. (2002); Bai and Bai (2014)
Ultimate Limit State (ULS)	$\varepsilon = \min(0.1, 1.4 \times t/D)$	A 'controllable' release of the containment of the pipeline is expected.	475	Bai (2001); Honegger et al. (2014); Bai and Bai (2014)
Global Collapse Limit State (GCLS)	$\varepsilon = 0.15$	A structural collapse is reported.	2475	Zhang (2008); Nazami and Das (2010); Ahmed et al. (2011); Bai and Bai (2014)

3
 4 The fragility curves are established based on the evolution of EDP , i.e. the peak axial strain of
 5 the pipeline ε in this study, with increasing earthquake intensity, encountering the associated
 6 uncertainties. In particular, PGV_{mi} are defined on the basis of relevant regression analyses of
 7 the axial strain of the pipeline ε with increasing PGV at ground surface. The latter is defined in
 8 this study as the maximum value of the peak values computed by the 1D soil response analyses
 9 of the adjacent subdeposits (see *Section 3.3*). It is worth noticing PGV has been used
 10 extensively as seismic IM in fragility relations for buried pipelines ([Bareberg 1988](#); [O'Rourke](#)
 11 [M.J. and Ayala 1993](#); [Eidinger et al. 1995](#); [Eidinger et al. 1998](#); [Jeon and O'Rourke T.D. 1995](#);
 12 [O'Rourke et al. 1998](#); [Isoyama et al. 2000](#); [ALA 2001](#); [Chen et al. 2002](#); [Pineda and Ordaz](#)
 13 [2003](#); [O'Rourke M.J. and Deyoe 2004](#); [Lanzano et al. 2013](#); [Lanzano et al. 2014](#); [Jahangiri and](#)
 14 [Shakib 2018](#)). The wide use of PGV is attributed to its direct relation with the longitudinal
 15 ground strain, which is responsible for the induced damages on buried pipelines caused by
 16 transient ground deformations. More importantly, in a recent study by [Tsinidis et al. \(2019c\)](#)
 17 this measure was found to be the most *efficient* and *proficient* one for the structural assessment
 18 of buried steel NG pipelines embedded in similar soils sites and subjected to similar seismic
 19 hazards. Finally, the metric satisfies the *hazard computability* criterion, since PGV hazard
 20 maps are commonly available after a major earthquake event.

21 With reference to the definition of the lognormal standard deviation, β_{tot} , which describes the
 22 total variability associated with each fragility curve; three primary sources of uncertainty are

1 considered (NIBS 2004) namely the definition of damage states, β_{ds} , the response and
2 resistance (capacity) of the element, β_C , and the earthquake input motion (demand), β_D . The
3 total uncertainty is estimated as the root of the sum of the squares of the component
4 dispersions. The uncertainty associated with the definition of damage states, β_{ds} , is set equal to
5 0.4, following HAZUS suggestions for buildings (NIBS 2004). In a similar manner, the
6 uncertainty due to the capacity, β_C , is assigned equal to 0.25. It is worth noticing that the
7 definition of both β_{ds} and β_C constitutes an open issue, particularly for embedded civil
8 infrastructure (Argyroudis and Pitilakis 2012, Argyroudis et al. 2017). A more rigorous
9 definition of the above parameters requires further detailed investigation, something that is
10 beyond the scope of the present study. The last source of uncertainty, associated with the
11 seismic demand, β_D , is described by the variability in response of the pipeline caused by the
12 variability of ground motion, and it is calculated as the dispersion of the simulated damage
13 indices with respect to the regression fit.

3.7 Limitations

16 Inevitably, there are some limitations of the analytical framework employed herein. The effects
17 of inertial SPI and of the evolution of stresses and deformations due to temperature changes on
18 the pipeline response, as well as time-dependent phenomena, such as fatigue and steel strength
19 and soil stiffness degradation due to cyclic loading, are neglected. Additionally, the
20 methodology does not account for all the sources that lead to spatial variability of the seismic
21 ground motion along the pipeline axis, which may further affect the response of the long buried
22 pipelines. 1D soil response analyses can not capture the potential 2D wave phenomena near the
23 geotechnical discontinuity or the generation of surface waves. However, 1D nonlinear soil
24 response analyses offer computational efficiency compared to 2D or 3D analyses and may be
25 used as a first approximation for the evaluation of the seismic response of the ground and
26 pipelines at shallow depths (Paolucci and Pitilakis 2007). The computational efficiency of 1D
27 soil response analyses allows for an extended and thorough parametric analysis, such as the
28 one presented in the ensuing. Finally, the methodology does not consider the potential effect of
29 the transversal seismic loading on the pipeline response.

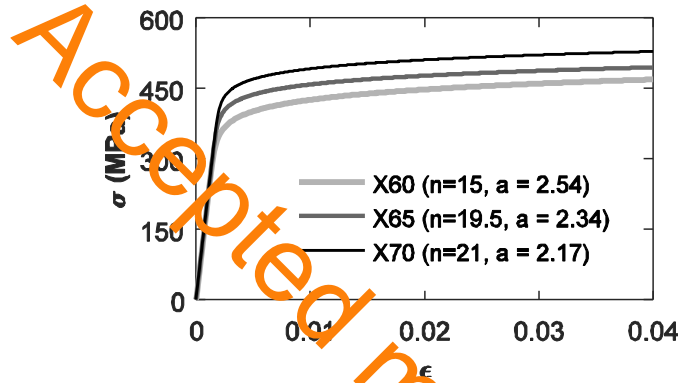
4. Numerical parametric study

32 A comprehensive numerical parametric study was conducted for various soil-pipe
33 configurations, employing the above analytical framework.

4.1 NG pipelines

36 The external diameter, D , and operational pressure, p , of the examined pipelines were selected
37 on the basis of a preliminary investigation of the variation of these characteristics in case actual
38 transmission NG networks found in several countries of Europe (Table 2). The external
39 diameter, D , wall thickness, t , and examined internal pressures, p , of the selected pipelines are
40 summarized in Table 3. The selected pipelines, which cover a wide range of diameter over
41 thickness ratios, D/t , that may be found in NG network applications, were designed following
42 the relevant regulations of ALA (2001) for a maximum operational pressure of $p = 9$ MPa. For

1 this maximum pressure level and by setting the external diameter of the pipeline, the wall
 2 thickness of the pipeline was calculated. Checks against ovaling due to earth loads were also
 3 carried out, as per ALA (2001). It was finally verified that the selected pipeline dimensions are
 4 available by the industry. The pipelines are made of API 5L X60, X65 and X70 grades, in an
 5 effort to cover a range of steel grades that are commonly used in this infrastructure. The
 6 mechanical properties of the selected grades are tabulated in Table 4, while Fig. 6 presents the
 7 axial stress-strain curves, which characterize the axial response of the examined pipelines and
 8 were defined by fitting Eq. 6 for a yield offset equal to 0.5 %. On this basis, the hardening
 9 exponents n are set equal to 15, 19.5 and 21, for grades X60, X65 and X70, respectively. The
 10 burial depth, h , of the selected pipelines, i.e. distance between the pipeline crown and ground
 11 surface, ranged between 1.0 m and 2.0 m, which constitute common burial depths for this
 12 infrastructure.



14
 15
 16 **Fig. 6** Uniaxial tensile stress-strain response of API X60, X65 and X70 steel grades adopted herein (n =
 17 hardening exponent, a = yield offset $\times E/\sigma_y$).

18
 19 **Table 2** External diameters and range of operational pressure of transmission NG pipeline networks
 20 found in several countries of Europe (information provided by the website of each operator).

Country	Operator	Nominal diameter range	Operational pressure range (MPa)
Austria	TAG	914.4 mm to 1066.8 mm (36' - 42')	7 - 8
Belgium	Fluxys Belgium	914.4 mm, 965.2 mm, 1016.0 mm (36', 38', 40')	4 - 7
Germany	Gascade	> 1066.8 mm (42') for the supra-regional networks; otherwise > 508 - 762 mm (20' - 30')	n.p*
Germany	Gasunie	> 1066.8 mm (42') for the supra-regional networks; otherwise > 508 - 762 mm (20' - 30')	n.p
Greece	DESFA	254 mm, 508 mm, 609.6 mm, 762 mm, 914.4 mm (10', 20', 24', 30', 36')	7
Italy	SNAM	508 - 1219.2 mm (20' to 48')	7 - 8
Spain	Enegas	406.4 - 812.8 mm (16' to 32')	n.p
Sweden	Swedegas	406.4 - 660.4 mm (16' to 26')	5 - 8
Switzerland	Transitgas	914.4 - 1066.8 mm (36' to 48')	7 - 8

* n.p.: not provided

21
 22
 23

1 **Table 3** Dimensions of examined pipes.

External diameter D (°)	External diameter D (mm)	Wall thickness t (mm)	D/t	R/t	Internal pressure, p (MPa)
16	406.4	9.5	42.8	21.4	0, 4, 8
20	508.0	8.7	58.4	29.2	
30	762.0	14.3	53.3	26.5	
36	914.4	12.7	72.0	36.0	
42	1066.8	15.9	67.1	33.55	
48	1219.2	19.1	63.8	31.9	

2
3 **Table 4** Mechanical properties of steel grades used in this study.

Steel grade	X60	X65	X70
Yield stress, σ_y (MPa)	414	448	483
Ultimate stress, σ_u (MPa)	517	531	565
Ultimate tensile strain, ε_u (%)	14.2	13	11.2
Young's modulus, E (GPa)	210	210	210

4
5 **4.2 Selected soil sites and backfill of trenches**

6 The depth of the selected soil sites, H , ranged between 30 m, 60 m and 120 m (Fig. 1). Both
7 cohesive and cohesionless soil deposits were examined, with the properties of the examined
8 pairs of subdeposits varying, in order to cover a range of anticipated sites. As stated above, a
9 surficial layer of cohesionless material was considered in all examined cases. This layer,
10 resting upon the examined pairs of subdeposits, had a depth equal to 3.0 m. Additionally, all
11 the examined sites were assumed to rest on an elastic bedrock with mass density, $\rho_b = 2.2 \text{ t/m}^3$
12 shear wave velocity $V_{s,b} = 1000 \text{ m/s}$.

13 With reference to the mechanical and physical properties of the subdeposits beneath the
14 surficial layer; Fig. 7 illustrates the gradients of the shear wave propagation velocities and the
15 mass densities, ρ , of the selected soil subdeposits. The variation of the small-strain shear
16 modulus of the cohesionless subdeposits follows the Seed and Idriss (1970) empirical formula:

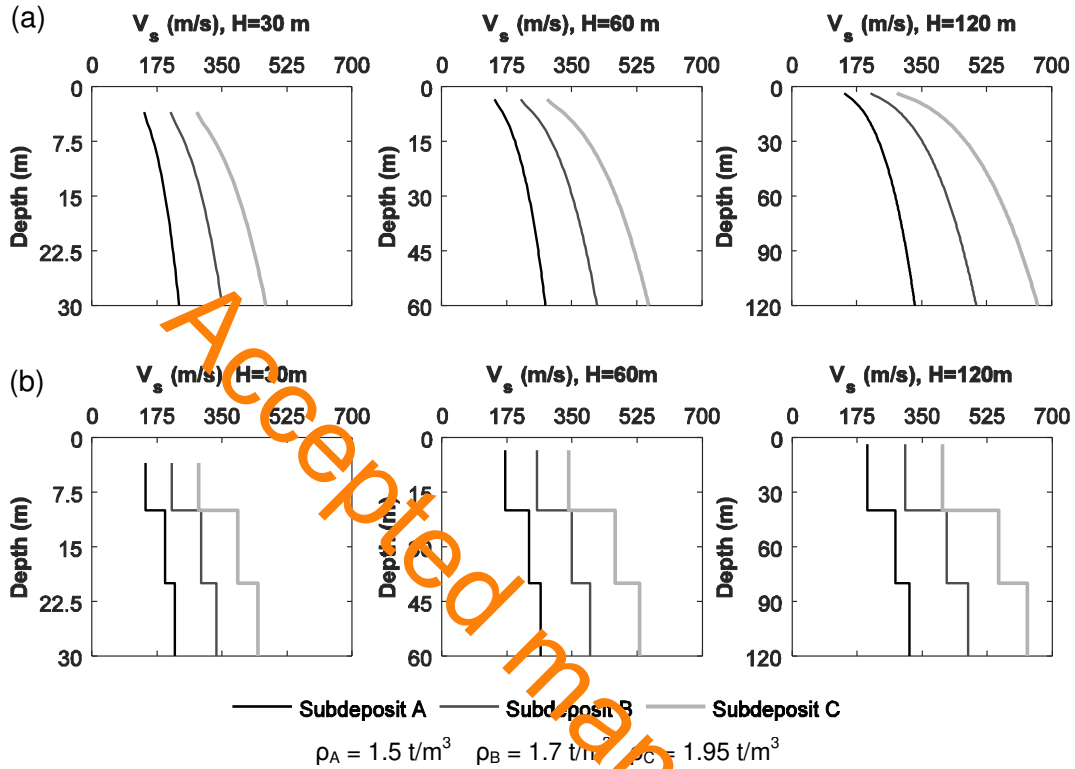
17
$$G_{\max} = 220K_{2,\max} (\sigma'_m)^{0.5} \quad (9)$$

18 where σ'_m is the mean effective confining stress and $K_{2,\max}$ is a constant depending on the
19 relative stiffness D_r of the sub-deposit (Table 5). Using Eq. 9 for the selected soil mass
20 densities and following basic elasto-dynamics, the gradients of small-strain shear wave
21 velocity were defined (Fig. 9a). The gradients of small-strain shear wave velocity of the
22 cohesive soil subdeposits were also considered to increase with depth (Fig. 9b). The selected
23 soil deposits correspond to soil classes B and C according to Eurocode 8 (CEN 2004). The
24 adopted profiles were selected in pairs, in order to define subdeposits 1 and 2 (Fig. 1). In
25 particular, three pairs were examined, i.e. Soil A - Soil B, Soil A - Soil C and Soil B - Soil C.
26 Accounting for the cohesionless and cohesive subdeposits, as well the diverse depths H of the
27 deposits, adopted herein, a total of 18 different cases was finally examined. The nonlinear
28 response of the all selected soil deposits during ground seismic shaking was described by
29 means of adequate G - γ - D curves provided by Darendeli (2001), which were employed in the
30 1D soil response analyses carried out in DEEPSOIL.

1 **Table 5** Relationships between density, relative density, $K_{2,max}$ parameter and cohesionless soil
 2 characterization (after Seed and Idriss 1970).

Density, ρ (t/m^3)	Relative density, D_r (%)	$K_{2,max}$	Characterization
1.4	30	30	Loose
1.65	52.5	48	Medium
2	90	70	Fine

3



4
5

6 **Fig. 7** Shear wave velocity gradients of examined (a) cohesive and (b) cohesionless soil sub-deposits.

7

8 Two different sets of mechanical and physical properties were examined for the surficial soil
 9 layer. This layer constitutes the trench backfill material for the examined pipelines and
 10 therefore is referred as either *trench TA* or *trench TB* in the ensuing, for the sake of simplicity.
 11 The selected properties, summarized in **Table 6**, correspond to well or very well-compacted
 12 conditions. For these upper bounds of backfill compaction level, an increased pipeline axial
 13 straining is expected under transient ground deformations. It is worth noting that the shear
 14 moduli, G , presented in **Table 6**, correspond to ‘average’ equivalent soil stiffnesses, referring to
 15 the ground strain range anticipated for the selected seismic ground motions, and were estimated
 16 on the basis of 1D soil response analyses, discussed in *Section 3.3*.

17

18 **Table 6** Physical and mechanical properties of investigated trenches.

	Density, ρ (t/m^3)	Poisson's ratio, ν	Shear modulus, G (MPa)	Friction angle, ϕ ($^\circ$)	Friction coefficient, μ
Trench TA	1.65	0.3	37.1	35	0.45
Trench TB	1.9	0.3	63.15	44	0.78

19

1 With reference to the selection of the friction coefficient of the trench backfill-pipe interface, μ ;
2 this may vary along the axis of a long-buried pipeline, while it may also change during ground
3 seismic shaking. However, it is bounded between the following limits, $\mu_{min}=0.3$ and $\mu_{max}=0.8$.
4 These limits are actually derived from the linear relation between the friction coefficient μ of
5 the soil-pipe interface and friction angle φ of the trench backfill soil, i.e. $\mu=(0.5-0.9)\times\tan\varphi$,
6 which was proposed by O'Rourke MJ & Hmadi (1988) and is commonly adopted in practice
7 (ALA 2001). For typical trench backfills the soil friction angle may range between 29° and 41° -
8 44° , yielding to above limits for the friction coefficients. The herein adopted friction
9 coefficients are presented in Table 6.

11 4.3 Seismic ground motions

12 An ensemble of 40 real ground motions is used in this study (Table 7), following a relevant
13 selection made by Fotopoulou and Pitilakis (2015), aiming for a diverse, yet unbiased sample
14 of ground motions. The motions were retrieved from the SHARE database (Giardini et al.
15 2013). The corresponding earthquake moment magnitudes M_w are varying between 5 and 7.62,
16 while the epicentral distances R , are ranging between 3.4 and 71.4 km. The motions were
17 recorded on rock outcrop or very stiff soil (soil classes A and B according to Eurocode 8, CEN
18 2004), with shear wave velocity of first 30 m, $V_{s,30}$, ranging between 650 m/s and 1020 m/s.
19 Note that outcrop motions were selected since they were to applied at the bedrock level of the
20 conducted 1D analyses. The input peak ground acceleration PGA varies between 0.065 g and
21 0.91 g, while the peak ground velocity PGV ranges between 0.031 m/s and 0.785 m/s.

23 5. Results and discussion

24 5.1 Verification of the 3D SPI model

25 The stresses and strains of a representative pipeline predicted under a particular axial ground
26 deformation pattern by implemented the 3D SPI model discussed above, are compared with
27 relevant results of an equivalent 'infinitely' long 3D continuum model of the soil-pipe
28 configuration subjected to the same kinematic loading condition, to verify the efficiency of the
29 selected 3D model. The length of the long 'infinite' model is set equal to 1000 times the
30 external diameter of the pipeline, D , to ensure that its predictions are approaching those of a
31 numerical model with 'infinite' length. In particular, the verification is carried out for a
32 'perfect' 914.4 mm pipeline, embedded at a burial depth, $h = 1.0$ m in both adopted surficial
33 soil-trenches. The pipeline is pressurized at an internal pressure $p = 8$ MPa. The nonlinear
34 springs that are introduced at the sides of the examined pipeline, as per Fig. 3, are initially
35 defined, as presented in Fig. 8. Fig. 8a illustrates the numerical model used to simulate the
36 axial pull-out of the pipeline. The pull-out analyses are performed examining both adopted
37 surficial soil-trenches, i.e. TA and TB (Table 6). The analyses yield the shear stress-
38 displacement relations provided in Fig. 8b. These relations are used to define the maximum
39 shear resistance τ_{max} and the shear stiffness k_s of the trench-pipe interface, which are
40 subsequently used to define the nonlinear springs, employing Eq. (1). The computed nonlinear

1 springs are presented in Fig 8c. Evidently, a higher friction coefficient for the trench-pipe
 2 interface leads to ‘stiffer’ springs.

3

4 **Table 7.** Selected records used in this study.

Date	Earthquake	Country	Station Name	Mw	R (km)	Preferred FS
25/07/2003	N Miyagi Prefecture	Japan	Oshika	6.1	32.00	Reverse
23/10/2004	Mid Niigata Prefecture	Japan	Tsunan	6.6	36	Reverse
12/06/2005	Anza	USA	Pinyon Flat Observatory	5.2	11.50	Strike-Slip
22/12/2003	San Simeon	USA	Ca: San Luis Obispo; Rec Center	6.4	61.5	Reverse
16/09/1978	Tabas	Iran	Tabas	7.35	57	Oblique
10/06/1987	Kalamata (Aftershock)	Greece	Kyparrisia-Agriculture Bank	5.36	17.00	Oblique
13/05/1995	Kozani	Greece	Kozani	6.61	17	Normal
07/09/1999	Ano Liosia	Greece	Athens 4 (Kipseli District)	6.04	17.00	Normal
15/04/1979	Montenegro	Serbia	Hercegnovi Novi-O.S.D.	6.9	65	Thrust
25/10/1984	Kremidia (Aftershock)	Greece	Peleanada-Town Hall	5	16	-
17/05/1995	Kozani (Aftershock)	Greece	Chromio-Community Building	5.3	16.00	Normal
13/10/1997	Kalamata	Greece	Koroni-Town Hall (Library)	6.4	48	Thrust
06/05/1976	Friuli	Italy	Tolmezzo-Diga Ambiesta	6.4	21.70	Reverse
15/09/1976	Friuli*	Italy	Tarcento	5.9	8.50	Reverse
23/11/1980	Irpinia	Italy	Bisaccia	6.9	28.30	Normal
14/10/1997	Umbria Marche*	Italy	Norcia	5.6	20.00	Normal
09/09/1998	App. Lucano	Italy	Lauria Galdo	5.6	6.60	Normal
06/04/2009	L Aquila Mainshock	Italy	L Aquila - V. Aterno - Colle Grilli	6.3	4.40	Normal
09/02/1971	San Fernando	USA	Lake Hughes #12	6.61	20.04	Reverse
28/11/1974	Hollister-03	USA	Gilroy Array #1	5.14	11.08	Strike-Slip
06/08/1979	Coyote Lake	USA	Gilroy Array #6	5.74	4.37	Strike-Slip
02/05/1983	Coalinga-01	USA	Slack Canyon	6.36	33.52	Reverse
24/04/1984	Morgan Hill	USA	Gilroy Array #6	6.19	36.34	Strike-Slip
23/12/1985	Nahanni, Canada	Greece	Site 1	6.76	6.8	Reverse
14/11/1986	Taiwan Smart1(45)	Taiwan	Smart1 E02	7.3	71.35	Reverse
07/02/1987	Baja California	USA	Cerro Prieto	5.5	3.69	Strike-Slip
18/10/1989	Loma Prieta	USA	Gilroy Array #6	6.93	35.47	Reverse-Oblique
18/10/1989	Loma Prieta	USA	Ucse Lick Observatory	6.93	16.34	Reverse-Oblique
25/04/1992	Cape Mendocino	USA	Petrolia	7.01	4.51	Reverse
28/06/1992	Landers	USA	Lucerne	7.28	44.02	Strike-Slip
17/01/1994	Northridge-01	USA	La - Griffith Park Observatory	6.69	25.42	Reverse
17/01/1994	Northridge-01	USA	Pacoima Dam (Downstr)	6.69	20.36	Reverse
16/01/1995	Kobe, Japan	Japan	Nishi-Akashi	6.9	8.7	Strike-Slip
20/09/1999	Chi-Chi, Taiwan	Taiwan	Tcu071	7.62	15.42	Reverse-Oblique
28/06/1991	Sierra Madre	USA	Mt Wilson - Cit Seis Sta	5.61	6.46	Reverse
16//10/1999	Hector Mine	USA	Hector	7.13	26.53	Strike-Slip
20/09/1999	Chi-Chi, Taiwan-03	Taiwan	Tcu129	6.2	18.5	Reverse
17/08/1999	Izmit	Turkey	Gebze-Tubitak Marmara	7.6	42.77	Strike-Slip
17/08/1999	Izmit	Turkey	Izmit-Meteoroloji Istasyonu	7.6	3.40	Strike-Slip

12/11/1999	Duzce 1	Turkey	Ldeo Station No. C1058 Bv	7.1	15.60	Strike-Slip
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* Aftershock

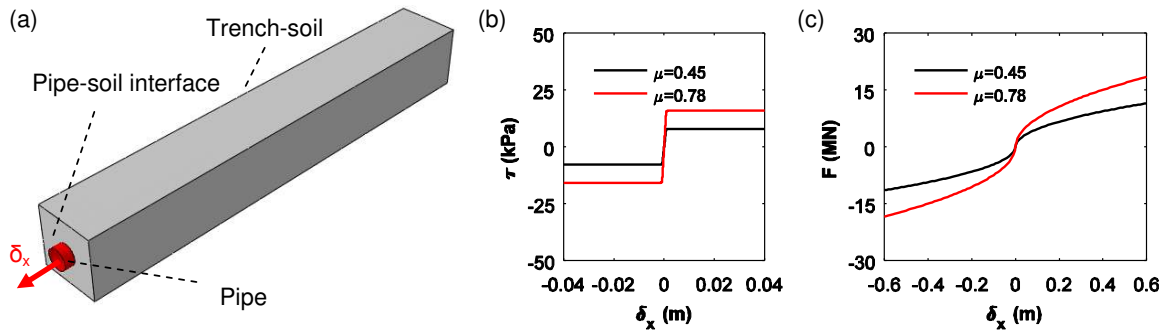
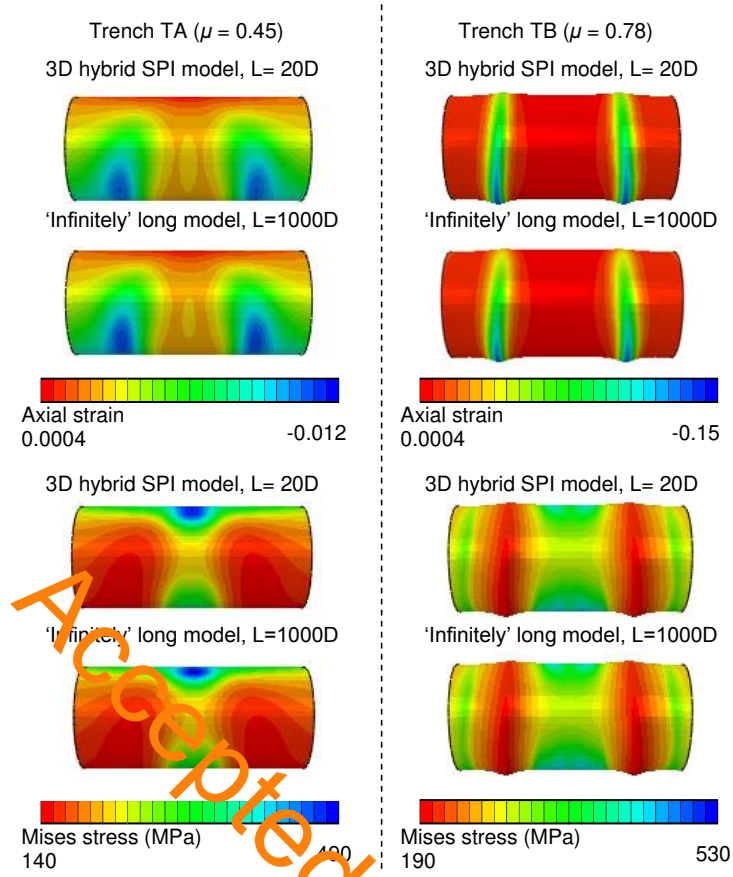


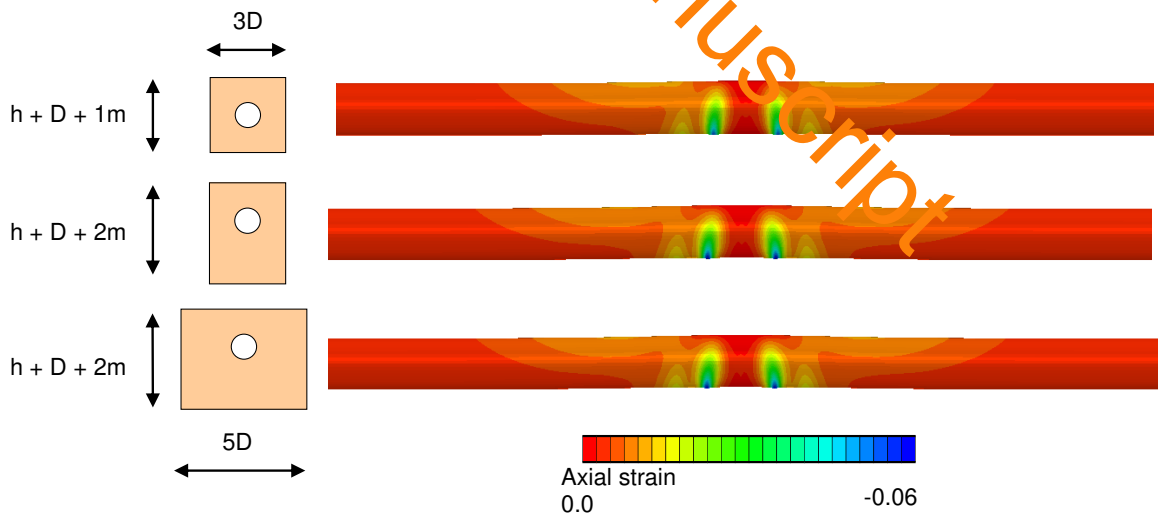
Fig. 8 (a) Numerical simulation of a buried pipeline subjected to axial pull-out, (b) shear stress–displacement relationship at the pipe-soil interface computed for the $D = 914.4$ mm pipeline, embedded in trench soil TA ($\mu = 0.45$) and TB ($\mu = 0.78$) in burial depth $h = 1.0$ m, (c) force-displacement relation of the nonlinear springs computed for the $D = 914.4$ mm pipeline, embedded in trench soil TA ($\mu = 0.45$) and TB ($\mu = 0.78$), $h = 1.0$ m.

Fig. 9 compares contour diagrams of the Mises stresses and axial strains computed at the critical central section of the pipeline by the two numerical models for a relative axial ground deformation $\delta_u = 20$ cm. The reduced-length 3D SPI model with the nonlinear spring at the sides of the pipeline provides quite similar –if not identical– results with the extended-length 3D SPI model, both in terms of stresses and strains, irrespectively of the assumed trench properties. Naturally, a higher axial response of the pipeline is reported for surficial soil-trench TB, where a rougher soil-pipe interface is considered. Evidently the computational cost of the reduced length model is significantly lower than that of the extended model. It is worth noticing that similarly good comparisons between the predictions of the reduced length 3D model with the nonlinear springs at both ends and the ‘infinitely’ long 3D model were observed, even when a geometric imperfection was considered on the central critical sections of the pipeline.

Fig. 10 elaborates on the effect of the selected width and depth of the trench soil model, surrounding the pipeline, by comparing contour diagrams of the axial straining computed on an examined pipeline, for different widths and depths of the trench continuum model. The results refer to a 762 mm ‘perfect’ pipeline embedded in surficial soil-trench TA at a burial depth $h = 1.0$ m. The pipeline is pressurized to an internal pressure $p = 8$ MPa and is subjected to a relative axial ground deformation $\delta_u = 20$ cm. The length of the models remains constant in all the examined cases, while nonlinear springs are properly defined as per **Fig. 8** for each examined ‘trench dimensions’. For the displacement loading patterns examined herein, the differences on the predictions of the pipeline axial strains are very small, if not negligible.



1
 2 **Fig. 9** Comparisons of contour diagrams of the Mises stresses and axial strain distributions computed at
 3 the critical central section of a 914.4 mm pipeline, embedded in backfill TA or TB, by the 3D SPI
 4 model with the nonlinear springs at both end sides and an 'infinitely' long equivalent 3D SPI model.
 5



6
 7 **Fig. 10** Contour diagrams of axial strains distributions computed on 762 mm 'perfect' pipeline
 8 embedded in trench TA, pressurized to an internal pressure $p = 8$ MPa and subjected to a relative axial
 9 ground deformation $\delta_u = 20$ cm, using 3D trench-soil models of diverse widths and depths.
 10
 11

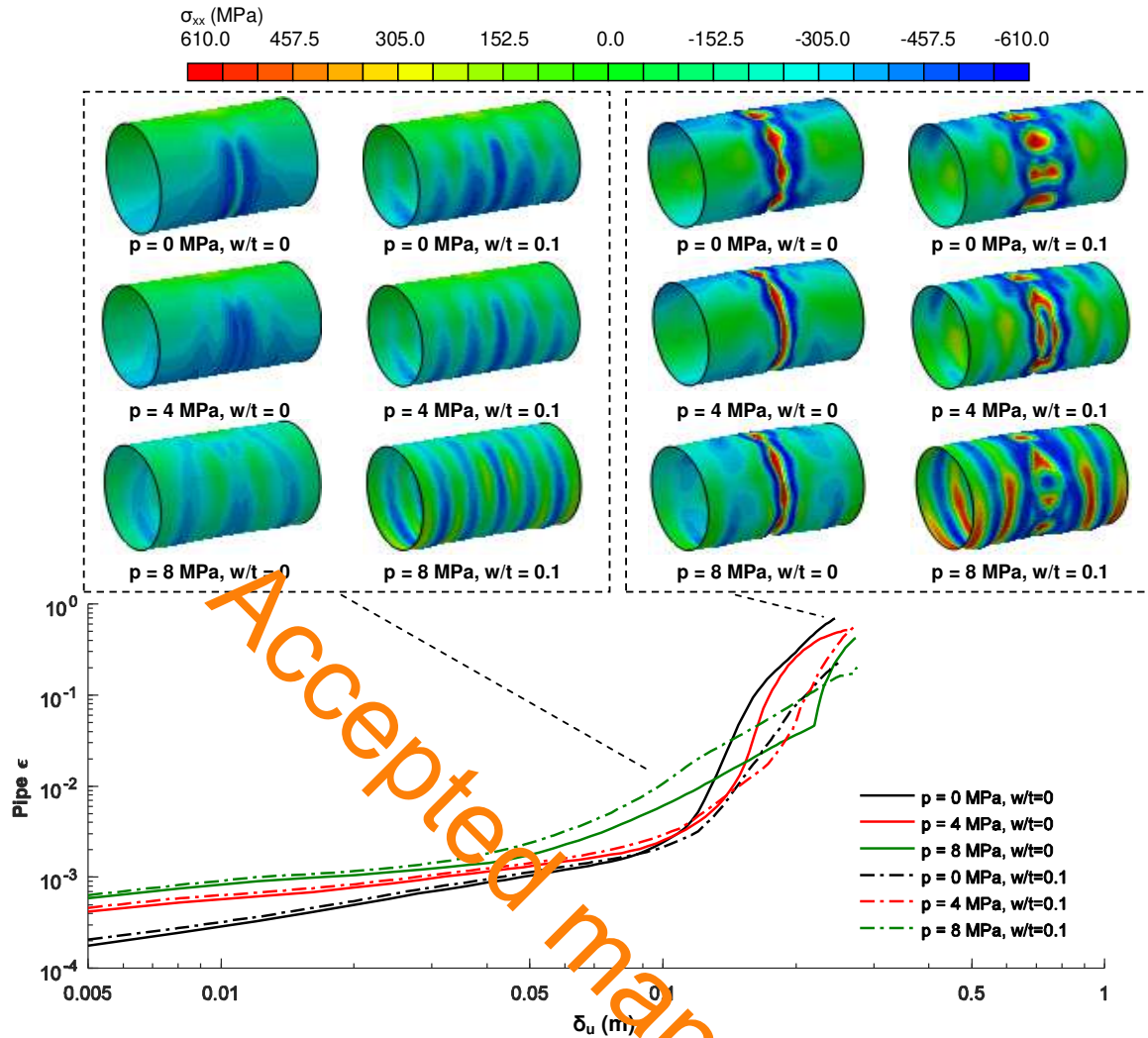
5.2 Pipeline response under an increasing relative axial deformation of the surrounding ground

Fig. 11 illustrates some representative results from 3D SPI analyses of a 1219.2 mm pipeline, embedded in surficial soil-trench TB at a burial depth, $h = 1.0$ m, elaborating on the effect of salient parameters on the axial response of embedded steel pressurized pipelines under an increasing relative axial ground deformation. In particular, contour diagrams of the axial stresses, developed at the critical zone of the pipeline, are plotted for two distinct steps of the analysis, i.e. before major concentration of stresses and buckling failure at the zone and at the end of the analysis, after buckling failure occurrence (end of analysis). The diagrams are plotted on the deformed shapes of the pipelines, so that to highlight the form of buckling failures that occur for higher levels of imposed relative axial ground deformations. Additionally, the figure portrays the evolution of maximum compressive strain of the critical pipeline zone with increasing relative axial ground deformation δ_u . The results are provided for various levels of internal pressure for the pipeline (i.e. $p = 0, 4$ MPa and 8 MPa), considering both a 'perfect' pipeline (i.e. $w/t=0$) and an equivalent imperfect pipeline (i.e. $w/t=0.1$). Evidently, both the pressurization level of the pipeline and the initial geometric imperfections of the pipeline wall affect the axial response of the examined pipelines.

In particular, with increasing relative axial ground deformation δ_u , the pipeline tends to bend upwards, i.e. towards the free ground surface. This response results in an early concentration of compressive axial stresses at the invert part of the pipeline, i.e. ditch axis of the pipeline. The existence of geometric imperfections is found to affect the distribution of the axial stresses on the pipeline. Actually, these stresses tend to distribute more uniformly across the invert of the perfect pipeline. On the contrary concentrations of stresses are observed at the imperfection 'bulges' of the imperfect pipeline.

The pressurization level tends to affect the buckling patterns of the examined pipelines, which take place under large relative axial ground deformations, i.e. $\delta_u \approx 12-20$ cm for the examined cases. Inward deformations of the pipe walls (i.e. deformations towards the pipe cavity) are observed for the non-pressurized (i.e. $p = 0$ MPa) or the low pressurized (i.e. $p = 4$ MPa) pipelines, while a combination of inward and outward deformations (i.e. deformations towards the trench soil) are observed on the highly pressurized pipelines (i.e. $p = 8$ MPa).

The effects of the wall imperfections and internal pressure are also evident on the evolution of maximum compressive axial strain of the critical zone of the pipeline with the increasing relative axial ground deformation δ_u . Higher strains are reported on the pressurized pipelines even at low δ_u , compared to those predicted on the non-pressurized pipelines. This observation is related to the combined effects of the operational internal pressure and axial compression of the pipeline caused by the seismic ground movement, on axial response of a steel pipeline (Paquette and Kyriakides 2006; Kyriakides and Corona 2007; Tsinidis et al. 2018). Additionally, the pipeline with the wall imperfection tends to concentrate higher strains throughout the analysis compared to the equivalent 'perfect' pipeline, with the differences between the two cases being as high as 18 %.

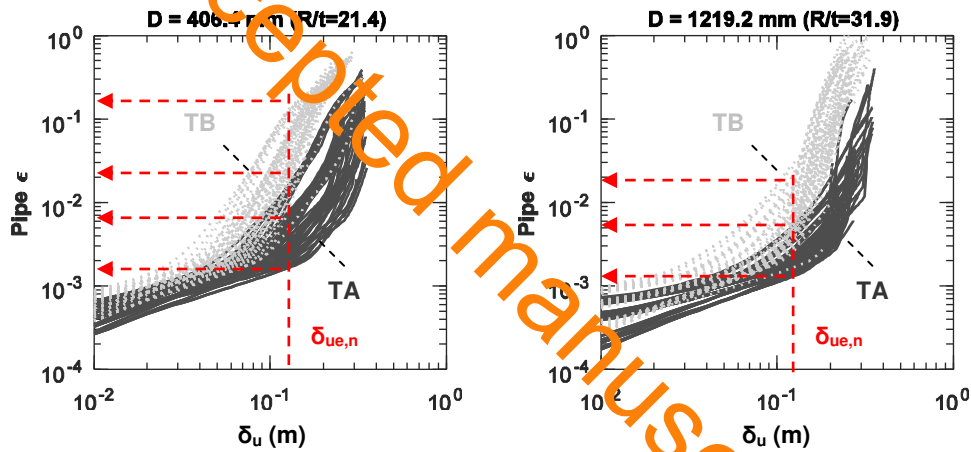


1
2 **Fig. 11** Effects of internal pressure and geometric imperfection of the pipe walls on the axial stresses
3 and the evolution of the maximum compressive strain, computed at the critical pipeline section for an
4 increasing relative axial ground deformation δ_u (results for a $D = 1219.2$ mm pipeline, embedded in
5 trench TB at a burial depth, $h = 1.0$ m).

6
7 **Fig. 12** presents representative comparisons of relations of maximum pipeline compressive
8 strain, ϵ , with increasing relative axial ground deformation δ_u , as computed by 3D SPI
9 analyses. The relations are actually plotted for 406.4 mm and 1219.2 mm ‘perfect’ or
10 ‘imperfect’ pipelines embedded at various depths, h , in surficial backfill soils TA and TB and
11 pressurized to various levels, i.e. $p = 0, 4, 8$ MPa. The comparisons highlight the critical effects
12 of pipeline dimensions, backfill properties and compaction level and backfill-pipe interface
13 friction characteristics on the axial response of the steel pipelines, induced by seismically-
14 induced relative axial ground deformations. Evidently, higher axial compression strains, ϵ , are
15 reported for the pipelines embedded in backfill TB. The higher shear stresses that are
16 developed along the ‘rougher’ backfill-pipe interface (the friction coefficient μ is equal to 0.78
17 in this case), result in an increased axial straining of the pipelines embedded in these surficial
18 soil conditions, compared to the equivalent pipelines embedded in trench TA. Additionally, the
19 higher confinement that is being offered by the surrounding ground on the pipeline, as a result

1 of its higher compaction level and stiffness, partially reduces the upward bending of the
 2 pipeline during the kinematic loading of the system (i.e. bending towards the ground surface,
 3 see Fig. 9), which in turn leads to an increased localization of axial straining at the critical zone
 4 of the pipeline, near the geotechnical discontinuity. Additionally, the concentration of axial
 5 compression strains, which subsequently leads to buckling of the pipelines, occurs in higher
 6 levels of relative axial ground deformation, δ_u , in the case of 1219.2 mm pipelines with thicker
 7 walls. It is worth noticing the wide range of pipeline strain ϵ that might be computed for a
 8 given level of relative axial ground deformation, δ_u , under various assumptions regarding the
 9 initial geometric imperfections of the pipeline walls, the backfill properties, the backfill-pipe
 10 interface characteristics and the internal pressure of the pipeline. Indeed for a given relative
 11 axial ground deformation $\delta_{ue,n}$ in Fig. 12 (associated with a ground motion and computed by the
 12 1D soil response analyses as per Section 3.3), a wide range of compressive strains ϵ may be
 13 computed on the pipeline, when considering all the above parameters. Hence, the consideration
 14 of these parameters is of great importance in the structural vulnerability assessment of this
 15 infrastructure.

16



17
 18 **Fig. 12** Evolution of the maximum compressive strain, ϵ , computed at the critical middle section of
 19 ‘perfect’ or ‘imperfect’ pipelines, embedded at various depths, h , in surficial backfill soils TA and TB
 20 and pressurized to various levels, i.e. $p = 0, 4, 8$ MPa, for an increasing relative axial ground
 21 deformation δ_u .

22

23 5.3 Fragility functions

24 Prior to the development of the fragility functions, the limit states defined in Section 3.5, are
 25 quantified for the selected pipelines, as per Table 1. Table 8, summarizes the limit axial
 26 compression strains for the four limit states for all examined pipelines. As seen, the maximum
 27 strain for OLS may range between 0.56 % and 0.94% for the examined pipelines. The range of
 28 strains for PILS varies between 2.4 % and 4 %, while for ULS the limit strain ranges between
 29 6.1 % and 10 %.

30

31

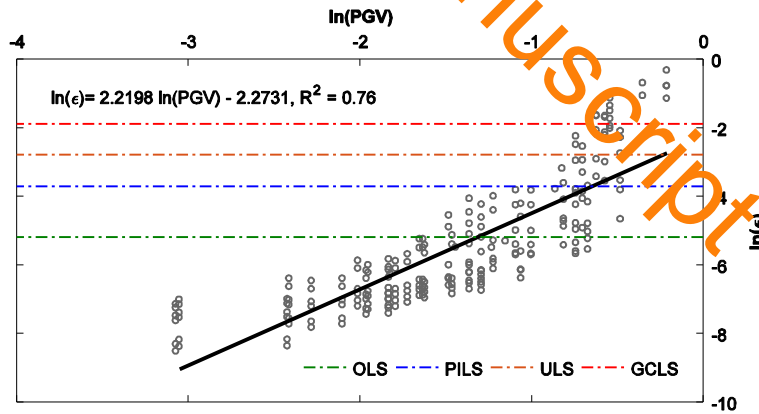
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33

1 **Table 8** Quantification of the maximum compression axial strains defined for each limit state and
 2 examined pipeline.

Diameter, D (mm)	406.4	508	762	914.4	1066.8	1219.2
Wall thickness, t (mm)	9.5	8.7	14.3	12.7	15.9	19.1
R/t	21.4	29.2	26.6	36.0	33.5	31.9
Maximum strain for limit state OLS, ϵ_{OLS}	0.0094	0.0069	0.0075	0.0056	0.0060	0.0063
Maximum strain for limit state PILS, ϵ_{PILS}	0.0400	0.0301	0.0330	0.0244	0.0262	0.0276
Maximum strain for limit state ULS, ϵ_{ULS}	0.1000	0.0754	0.0826	0.0611	0.0656	0.0689
Maximum strain for limit state GCLS, ϵ_{GCLS}	0.1500	0.1500	0.1500	0.1500	0.1500	0.1500

3
 4 Having quantified the maximum strain for each limit state and examined pipeline, regression
 5 analyses of the natural logarithm of the computed maximum axial strain ϵ of the examined
 6 pipeline relative to the natural logarithm of the peak ground velocity PGV at ground surface are
 7 carried out, as per Fig. 13. The regressions are actually conducted for each soil-pipe
 8 configuration, by combining the numerical predictions of the above analytical framework,
 9 referring to various levels of internal pressure for pipelines (i.e. $p = 0, 4$ and 8 MPa) and
 10 various assumptions regarding the initial geometric imperfection of the pipeline walls (i.e.
 11 combining the results referring to $v/t = 0$ or $w/t = 0.1$). The selection is made on the ground
 12 that existence of geometric imperfections on pipeline walls is not a-priori known. Additionally,
 13 the use of results, referring to a range of internal pressures, allows for a general application of
 14 the provided fragility curves in networks with similar ranges of operational pressure. The
 15 results of the regression analyses are used to define the parameters that are required to
 16 construct the fragility functions, i.e. PGV_{mi} and β_{tot} .
 17



18
 19
 20 **Fig. 13** Example of evolution of $\ln(\epsilon)$, with earthquake intensity measure $\ln(PGV)$ and regression
 21 analysis used to define PGV_{mi} and β_{tot} (numerical results for of a 762 mm pipeline embedded in Trench
 22 TA at a burial depth, $h = 1.0$ m in soil deposit with depth $H = 30$ m)
 23

24 Based on the above procedure, an extended set of more than 1200 fragility functions was
 25 constructed, referring to diverse examined soil-pipe configurations. The relevant PGV_{mi} and β_{tot}
 26 for all the examined curves are summarized in a set of tables, summarized in Appendix A. In
 27 the following, some representative fragility curves are comparatively presented, aiming at

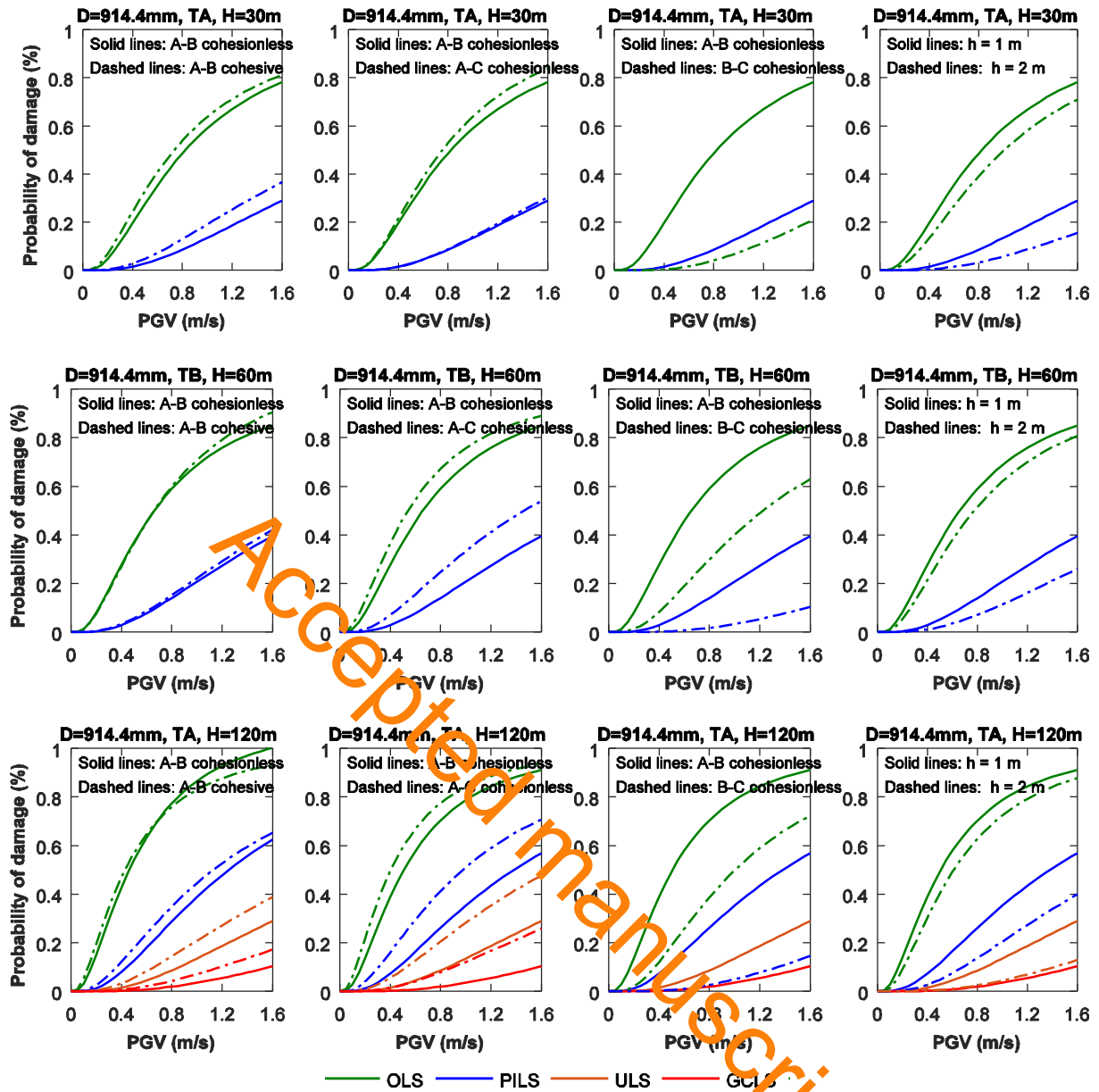
1 discussing the effects of salient parameters on the computed fragility of the examined NG
2 pipelines.

3 **Fig. 14** compares analytical fragility relations, referring to X65 914.4 mm pipelines embedded
4 in surficial soil-trench TA, highlighting the effects of the depth, H , of the soil deposit, the
5 subdeposits characteristics and pipeline burial depth h , on the seismic fragility of NG pipelines.
6 Generally, the seismic fragility of the examined pipelines is found to be low. Actually, the ULS
7 and GCLS limit states are not reached for the examined pipelines embedded in soil deposits
8 with depths, H , equal to 30 m and 60 m, while the seismic fragility of examined pipelines is
9 found to increase with increasing burial depth, H , of the soil deposit. This observation is
10 attributed to the higher differential ground response of the adjacent subdeposits, expected for
11 deposits with higher depths, i.e. for $H = 120$, compared to the one of deposits with depths, H ,
12 equal 30 m or 60 m, when subjected to the same excitation at the bedrock. The higher
13 differential ground response of the adjacent subdeposits induces a higher axial straining on the
14 pipeline, thus increasing the potential of damage under a given seismic intensity.

15 The seismic fragility of the examined pipelines embedded in cohesionless soils is found to be
16 slightly lower compared to the one predicted for the cases where the pipelines are embedded in
17 cohesive soils (see subplots on the left-hand side of **Fig. 14**). The differences are again
18 attributed to the distinct ground response characteristics of the examined soil subdeposits,
19 which induce distinct axial straining on the embedded pipelines.

20 The higher contrast on the soil properties of the adjacent soil subdeposits, leads naturally to a
21 more dissimilar response of the subdeposits, which induces a higher straining on the pipeline,
22 thus resulting in a higher potential for damage, under a given intensity. This hypothesis is
23 verified by comparing the fragility curves developed for the pairs of subdeposits A-B and A-C.
24 Indeed, a higher fragility is reported in case of pipelines embedded in soil with subdeposits A-
25 C, where the differences on the soil properties of the subdeposits (i.e. shear wave velocity and
26 mass density) are higher compared to the case of the soil that consists of subdeposits A-B.
27 Comparing the fragility of pipelines embedded in soils with subdeposits A-B and B-C, a much
28 higher fragility is reported in the former soil site, even though the level of contrast of the
29 properties of the adjacent subdeposits is the same for both cases. This is actually expected,
30 given the lower ground seismic response of ‘stiffer’ soil deposits, i.e. the soil with subdeposits
31 B-C in this case, compared to soft soil deposits (i.e. soil with subdeposits A-B), under a given
32 seismic intensity.

33 A higher fragility is systematically reported for pipelines embedded at a burial depth, $h = 1.0$
34 m, compared to the cases, where the equivalent pipelines are embedded deeper, i.e. $h = 2.0$ m.
35 This should be attributed to the higher horizontal ground movement of the soil deposits
36 towards the ground surface, during ground shaking, which causes higher relative ground
37 deformations on the shallow-buried pipelines, hence increasing their axial response and
38 damage potential.

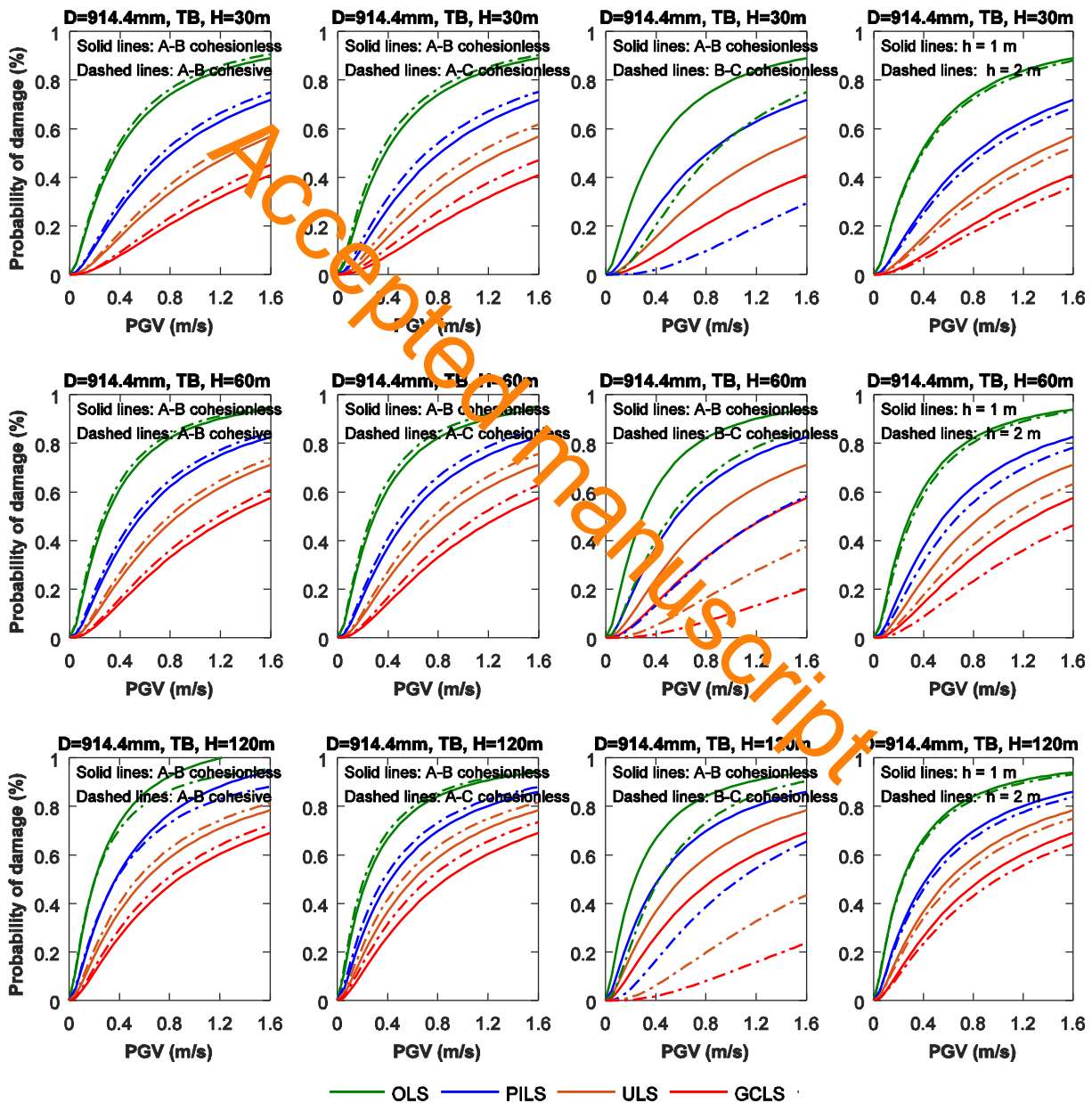


1
2
3 **Fig. 14** Fragility functions of X65 914.4 mm pipelines embedded in surficial soil-trench TA. Effects of
4 soil deposit depth, H , subdeposits characteristics and burial depth, h , of the pipelines on the seismic
5 fragility.

6
7 It is worth noticing that the general low fragility of the buried pipelines in surficial soil-trench
8 TA comes in line with the reported good performance of buried NG pipelines and their reduced
9 fragility reported during past earthquakes. It is recalled that the first two limit states, adopted in
10 this study, constitute operational limit states that do not lead to wall tearing and leakages and
11 basically do not affect the flow of containment - at least not significantly. This makes the
12 observation of relevant actual damages rather difficult after an earthquake event.

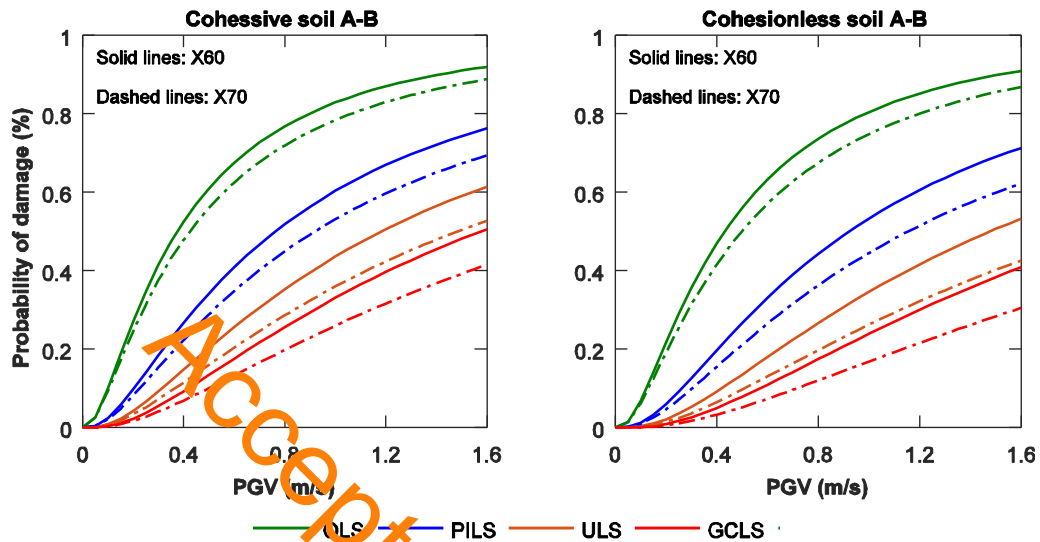
13 **Fig. 15** compares similar fragility relations, referring to the same pipelines, i.e. X65 914.4 mm
14 pipelines, embedded in surficial soil-trench TB. Evidently, a much higher fragility is reported
15 in this case, compared to the previous results. All limit states are reached, even for the cases
16 where the pipelines are embedded in the shallow soil deposits, i.e. for $H = 30$ m and 60 m. This

1 observation is in line with the higher axial compressive straining of the pipeline that is
 2 anticipated in cases of a dense backfill trench with a 'rougher' backfill-pipe interface (i.e.
 3 surficial soil-trench TB). In line with the previous results, the fragility of the examined
 4 pipelines is increased with increasing depth of the soil deposits, H , decreasing burial depth of
 5 the pipelines, h , and increasing contrast of the properties of the adjacent soil subdeposits. A
 6 slightly higher fragility is reported for the pipelines embedded in the soil deposits with
 7 cohesive subdeposits. It is worth noting that higher total lognormal standard deviations β_{tot}
 8 were estimated for the fragility relations referring to pipelines embedded in surficial soil-trench
 9 TB, associated with the generally amplified axial response of pipelines for these conditions.
 10



11
 12
 13 **Fig. 15** Fragility functions of 914.4 mm X65 pipelines embedded in surficial soil-trench TB. Effects of
 14 soil deposit depth, H , subdeposits characteristics and burial depth, h , of the pipelines on the seismic
 15 fragility.

1 The effect of the steel grade of the pipeline on its fragility is highlighted in Fig. 16, where
 2 fragility curves referring to 762 mm pipelines made of different steel grades, i.e. X60 and X70,
 3 are compared. The comparisons refer to pipelines embedded in surficial soil-trench TB in
 4 cohesive or cohesionless soil deposits A-B of depth $H = 60$ m. As expected, the fragility of
 5 X60 pipelines is found higher, compared to the equivalents made of X70 steel grade.
 6



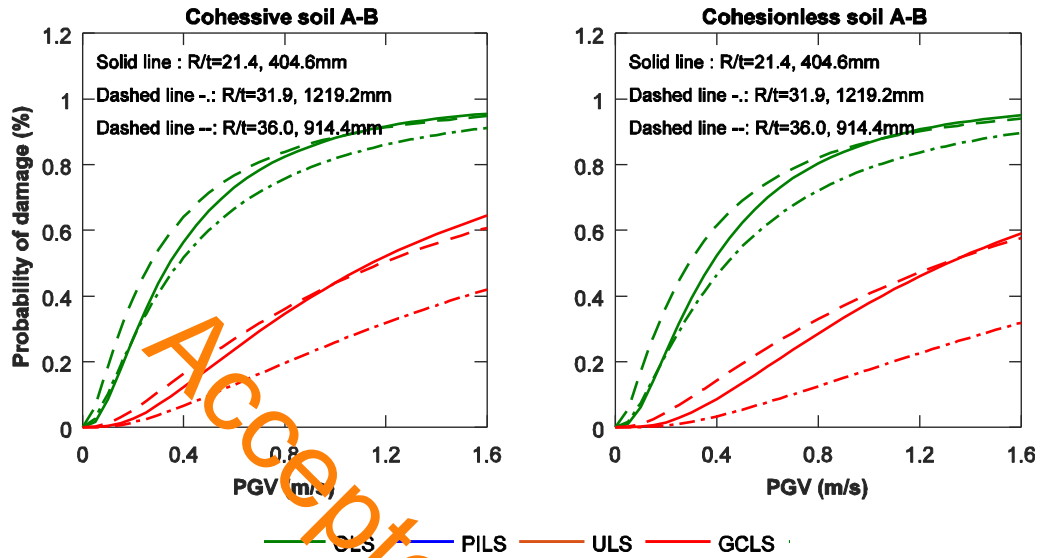
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 8
 9 **Fig. 16.** Effect of steel grade of pipelines on their seismic fragility. Fragility functions for 762 mm
 10 pipelines embedded in surficial soil-trench TB in cohesive or cohesionless soil deposits A-B of depth H
 11 = 60 m.

12
 13 Fig. 17 compares fragility relations, referring to pipelines with diverse dimensions, embedded
 14 in similar trench soil conditions and burial depth, h , (i.e. soil deposit A-B of depth $H = 60$ m
 15 and pipeline burial depth $h = 1.0$ m), in an effort to highlight the effect of the radius over
 16 thickness ratio (R/t) of the pipelines on their seismic fragility. The comparisons do not provide
 17 a clear trend between the radius over thickness ratio (R/t) of the pipeline and its seismic
 18 fragility. Indeed, the highest fragility is reported for the 914.4 mm pipelines with $R/t = 36$,
 19 followed by the 406.4 mm pipelines with $R/t = 21.4$. The lowest fragility is reported for the
 20 1219.2 mm pipelines with $R/t = 31.9$.

22 6. Conclusions

23 A detailed analytical framework was developed to evaluate the fragility of NG pipelines
 24 crossing sites with a vertical geotechnical discontinuity, when subjected to axial compression
 25 caused by transient seismic ground deformation. The methodological framework consists of a
 26 3D SPI model, aiming at evaluating the pipe-trench interaction effects on the pipeline axial
 27 response in quasi-static manner and 1D soil response analyses, used to determine critical
 28 ground deformation patterns at the geotechnical discontinuity under ground shaking. The
 29 efficiency of the 3D SPI model in replicating the actual soil-pipe interaction phenomena,
 30 associated with the extended length of this infrastructure was thoroughly verified. A
 31 comprehensive parametric analysis was performed, using the proposed analytical framework,

1 for an extended number of soil-pipe configurations and an ensemble of 40 recorded earthquake
 2 ground motions, which led to the definition of an extended set of fragility curves. The latter
 3 were developed in terms of PGV at the ground surface, for four performance limit states,
 4 considering the associated uncertainties. The main conclusions of the study are summarized in
 5 the following:



7
 8
 9 **Fig. 17.** Effect of radius over thickness (R/t) ratio of diverse pipelines, embedded in trench TB in
 10 cohesive or cohesionless soil deposits A-B of depth $H = 60$ m, on their seismic fragility.

- 11
- 12 • The seismic fragility of buried steel NG pipelines against seismically-induced axial
 13 compression near geotechnical discontinuities was found to be rather low, when the
 14 examined pipelines were embedded in a medium-compacted surficial layer with a medium
 15 friction coefficient μ being considered for the trench soil-pipe interface (i.e. surficial soil-
 16 trench TA in this study). Actually, for these cases, the ULS and GCLS limit states, which
 17 are associated with major failures or collapse were not reached in the majority of the
 18 examined soil-pipe configurations. This observation is in line with the reduced
 19 vulnerability of this infrastructure against transient ground deformations, reported during
 20 past earthquakes.
 - 21 • On the contrary, a higher fragility was reported for the pipelines embedded in a ‘stiffer’
 22 very well-compacted surficial soil-trench, with a high friction coefficient μ being
 23 considered for the trench soil-pipe interface (i.e. surficial soil-trench TB). This is mainly
 24 attributed to the higher axial straining of the pipeline, which is caused by the higher
 25 stresses developed along the trench-pipe interface during the surrounding ground
 26 deformation. Additionally, the higher compaction level and stiffness of the surrounding
 27 ground lead to a higher confinement of the pipeline, which reduces the bending of the
 28 pipeline towards the ground surface, during the kinematic loading of the system, thus
 29 increasing the local straining of the pipeline, finally contributing to an increased damage
 30 potential. In the light of the above observations, it is very important to avoid over-

1 compacting of the trench soil of buried NG pipelines, or even try to reduce the soil-pipe
2 interface friction, particularly in seismically-prone regions of varying soil conditions, in
3 order to reduce the fragility of this infrastructure under seismically-induced transient
4 ground deformations.

- 5 • Regardless of the trench backfill properties and backfill-pipe interface characteristics the
6 seismic fragility of the examined NG pipelines was increased with increasing depth, H , of
7 the soil deposit. This was attributed to the higher differential ground response of deeper
8 adjacent subdeposits, e.g. for $H = 120$ m, under a given excitation at bedrock. The higher
9 differential ground response of the adjacent subdeposits induced a higher axial straining on
10 the pipeline, thus increasing the potential of damage.
- 11 • The seismic fragility of pipelines was found to be slightly lower for the cohesionless
12 subdeposits, studied herein, compared to the one predicted for equivalent pipelines in
13 cohesive subdeposits.
- 14 • The higher contrast on the soil properties of the adjacent soil subdeposits, led to a more
15 dissimilar seismic response of the subdeposits under a given excitation at bedrock, which
16 subsequently led to a higher axial straining for the embedded pipeline and hence to a higher
17 damage potential.
- 18 • A higher fragility was systematically reported for pipelines embedded at a burial depth, $h =$
19 1.0 m, compared to the cases, where the equivalent pipeline was embedded at a higher
20 depth, i.e. $h = 2.0$ m. Naturally, pipelines made of X60 steel grade were found to be more
21 vulnerable to the particular seismic hazard, compared to those made of higher steel grades,
22 e.g. X65 and X70, while it was not possible to define a clear trend between the radius over
23 thickness (R/t) of the pipeline and its seismic fragility.

24 Inevitably, there are some limitations of the analytical framework used herein. The effects of
25 inertial SPI and of the evolution of stresses and deformations due to temperature changes on
26 the pipeline response, as well as time-dependent phenomena, such as fatigue and steel strength
27 and stiffness degradation due to cyclic loading, are not considered in the present analysis
28 framework. Additionally, the proposed analysis method does not account for all the sources
29 that may lead to spatial variability of the seismic ground motion along the pipeline axis.
30 Moreover, complex 2D wave phenomena near the geotechnical discontinuity are not being
31 thoroughly investigated. However, the study covers a wide range of salient parameters that
32 may affect the response and vulnerability of buried steel NG pipelines, crossing similar sites.
33 In this context, the use of the provided analytical fragility curves may contribute towards a
34 more reliable quantitative risk assessment of buried steel NG pipelines, subjected to
35 seismically-induced transient ground deformations, particularly if combined with the practical
36 recommendations reported above.

38 **Acknowledgements**

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1 Hybrid Assessment of NG Pipelines Exposed to Seismic Hazard, www.exchange-risk.eu). This
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28 **Appendix A**

29 A series of tables, summarizing the parameters required for the definition of the fragility curves
30 developed in the framework of this study, i.e. the median peak ground velocities corresponding to the
31 limit states, $PGV_{m,i}$ and total lognormal standard deviation β_{tot} , are summarized in this appendix.

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1 **Table A1** Median peak ground velocities corresponding to the limit states, $PGV_{m,l}$, and total lognormal
2 standard deviation, β_{tot} , for 406.4 mm pipelines embedded in soil deposit of depth $H = 30$ m (- : the
3 limit state is not reached).
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			Cohesive soil deposit – Trench type TA					Cohesionless soil deposit – Trench type TA				
Soil	Steel Grade	Burial depth, h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.458	2.840	-	-	0.773	1.133	3.199	-	-	0.769
A-B	X65	1	1.167	3.407	-	-	0.749	1.303	3.955	-	-	0.740
A-B	X70	1	1.278	3.915	-	-	0.725	1.424	4.527	-	-	0.717
A-B	X60	2	1.952	-	-	-	0.642	2.274	-	-	-	0.631
A-B	X65	2	2.045	-	-	-	0.659	2.339	-	-	-	0.650
A-B	X70	2	2.341	-	-	-	0.625	2.681	-	-	-	0.615
A-C	X60	1	1.070	3.101	-	-	0.758	1.016	3.154	-	-	0.761
A-C	X65	1	1.249	3.886	-	-	0.734	1.284	4.049	-	-	0.725
A-C	X70	1	1.571	4.529	-	-	0.711	1.374	4.220	-	-	0.676
A-C	X60	2	2.176	-	-	-	0.629	2.296	-	-	-	0.623
A-C	X65	2	2.288	-	-	-	0.649	2.388	-	-	-	0.644
A-C	X70	2	2.647	-	-	-	0.612	2.750	-	-	-	0.607
B-C	X60	1	4.315	-	-	-	0.621	4.655	-	-	-	0.606
B-C	X65	1	4.296	-	-	-	0.612	-	-	-	-	-
B-C	X70	1	4.691	-	-	-	0.601	-	-	-	-	-
B-C	X60	2	-	-	-	-	-	-	-	-	-	-
B-C	X65	2	-	-	-	-	-	-	-	-	-	-
B-C	X70	2	-	-	-	-	-	-	-	-	-	-
			Cohesive soil deposit – Trench type TB					Cohesionless soil deposit – Trench type TB				
Soil	Steel Grade	h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.458	0.913	1.411	1.711	0.961	0.488	0.990	1.546	1.882	0.995
A-B	X65	1	0.516	1.031	1.596	1.936	0.973	0.513	1.079	1.714	2.103	1.019
A-B	X70	1	0.505	1.048	1.661	2.036	1.020	0.558	1.205	1.959	2.428	1.027
A-B	X60	2	0.486	0.990	1.551	1.892	1.114	0.551	1.197	1.953	2.426	1.085
A-B	X65	2	0.544	1.176	1.913	2.372	1.112	0.625	1.452	2.470	3.125	1.067
A-B	X70	2	0.621	1.445	2.461	3.114	1.066	0.715	1.791	3.194	4.125	1.009
A-C	X60	1	0.432	0.862	1.333	1.616	0.969	0.447	0.900	1.399	1.701	0.968
A-C	X65	1	0.449	0.911	1.422	1.731	1.011	0.473	0.978	1.544	1.891	0.999
A-C	X70	1	0.492	1.047	1.686	2.081	1.011	0.520	1.129	1.840	2.284	0.994
A-C	X60	2	0.482	1.019	1.632	2.011	1.088	0.512	1.118	1.830	2.275	1.055
A-C	X65	2	0.549	1.243	2.082	2.615	1.078	0.586	1.377	2.358	2.992	1.038
A-C	X70	2	0.568	1.252	2.062	2.571	1.077	0.603	1.370	2.299	2.891	1.045
B-C	X60	1	1.290	3.727	-	-	0.812	1.414	4.231	-	-	0.765
B-C	X65	1	1.305	3.780	-	-	0.811	1.709	-	-	-	0.767
B-C	X70	1	1.455	4.447	-	-	0.786	1.591	4.657	-	-	0.713
B-C	X60	2	1.440	4.427	-	-	0.834	2.047	-	-	-	0.749
B-C	X65	2	1.652	-	-	-	0.812	2.407	-	-	-	0.718
B-C	X70	2	1.908	-	-	-	0.770	2.845	-	-	-	0.671

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1 **Table A2** Median peak ground velocity corresponding to the limit states, $PGV_{m,i}$ and total lognormal
2 standard deviation β_{tot} for 406.4 mm pipelines embedded in soil deposit of depth $H = 60$ m (- : the limit
3 state is not reached).
4

			Cohesive soil deposit – Trench type TA					Cohesionless soil deposit – Trench type TA				
Soil	Steel Grade	Burial depth, h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.809	2.176	4.059	-	0.741	0.902	2.482	4.700	-	0.755
A-B	X65	1	0.924	2.660	-	-	0.719	1.013	2.949	-	-	0.724
A-B	X70	1	1.044	3.212	-	-	0.697	1.069	3.187	-	-	0.731
A-B	X60	2	1.525	-	-	-	0.634	1.811	-	-	-	0.630
A-B	X65	2	1.781	-	-	-	0.621	1.811	-	-	-	0.662
A-B	X70	2	2.081	-	-	-	0.593	2.144	-	-	-	0.618
A-C	X60	1	0.700	1.749	3.117	4.024	0.742	0.742	1.878	3.374	4.372	0.758
A-C	X65	1	0.797	2.106	3.904	-	0.723	0.845	2.287	4.286	-	0.738
A-C	X70	1	0.877	2.468	4.739	-	0.705	0.952	2.752	-	-	0.717
A-C	X60	2	1.245	1.194	-	-	0.646	1.376	4.867	-	-	0.652
A-C	X65	2	1.495	-	-	-	0.623	1.595	-	-	-	0.641
A-C	X70	2	1.744	-	-	-	0.598	1.859	-	-	-	0.610
B-C	X60	1	1.533	4.700	-	-	0.741	1.775	-	-	-	0.703
B-C	X65	1	1.724	-	-	-	0.700	2.172	-	-	-	0.674
B-C	X70	1	1.810	-	-	-	0.687	2.399	-	-	-	0.646
B-C	X60	2	2.895	-	-	-	0.611	3.422	-	-	-	0.614
B-C	X65	2	3.711	-	-	-	0.592	4.016	-	-	-	0.590
B-C	X70	2	3.711	-	-	-	0.577	4.588	-	-	-	0.575
			Cohesive soil deposit – Trench type TB					Cohesionless soil deposit – Trench type TB				
Soil	Steel Grade	h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.345	0.647	0.961	1.145	0.903	0.380	0.727	1.094	1.310	0.870
A-B	X65	1	0.354	0.665	0.989	1.180	0.935	0.395	0.766	1.162	1.397	0.892
A-B	X70	1	0.376	0.725	1.098	1.319	0.952	0.425	0.847	1.310	1.590	0.911
A-B	X60	2	0.366	0.697	1.048	1.255	1.051	0.410	0.915	1.260	1.526	1.008
A-B	X65	2	0.400	0.795	1.227	1.487	1.061	0.456	0.978	1.530	1.883	0.998
A-B	X70	2	0.439	0.918	1.462	1.796	1.059	0.522	1.185	1.983	2.492	0.964
A-C	X60	1	0.432	0.862	1.333	1.616	0.938	0.336	0.607	0.883	1.042	0.927
A-C	X65	1	0.324	0.578	0.833	0.979	0.961	0.341	0.616	0.893	1.053	0.960
A-C	X70	1	0.340	0.620	0.906	1.072	0.972	0.359	0.665	0.981	1.165	0.964
A-C	X60	2	0.330	0.591	0.854	1.005	1.070	0.351	0.652	0.962	1.143	1.045
A-C	X65	2	0.353	0.654	0.965	1.146	1.090	0.387	0.754	1.148	1.383	1.032
A-C	X70	2	0.389	0.760	1.158	1.395	1.080	0.427	0.880	1.387	1.696	1.018
B-C	X60	1	0.689	1.599	2.721	3.442	0.936	0.801	1.961	3.449	4.428	0.900
B-C	X65	1	0.733	1.744	3.014	3.839	0.956	0.828	2.055	3.644	4.696	0.929
B-C	X70	1	0.796	1.978	3.511	4.526	0.930	0.874	2.220	3.998	-	0.917
B-C	X60	2	0.796	1.989	3.544	4.575	0.985	0.809	1.984	3.492	4.485	1.003
B-C	X65	2	0.843	2.157	3.901	-	0.986	0.891	2.294	4.164	-	0.978
B-C	X70	2	0.966	2.663	-	-	0.920	1.000	2.736	-	-	0.924

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1 **Table A3** Median peak ground velocity corresponding to the limit states, $PGV_{m,l}$, and total lognormal
 2 standard deviation, β_{tot} , for 406.4 mm pipelines embedded in soil deposit of depth $H = 120$ m (- : the
 3 limit state is not reached).
 4

			Cohesive soil deposit – Trench type TA					Cohesionless soil deposit – Trench type TA				
Soil	Steel Grade	Burial depth, h (m)	PGV _{m,OLS} (m/s)	PGV _{m,PILS} (m/s)	PGV _{m,ULS} (m/s)	PGV _{m,GLS} (m/s)	β_{tot}	PGV _{m,OLS} (m/s)	PGV _{m,PILS} (m/s)	PGV _{m,ULS} (m/s)	PGV _{m,GLS} (m/s)	β_{tot}
A-B	X60	1	0.548	1.390	2.498	3.239	0.718	0.613	1.526	2.710	3.495	0.712
A-B	X65	1	0.591	1.549	2.842	3.718	0.721	0.680	1.784	3.275	4.286	0.691
A-B	X70	1	0.657	1.817	3.451	4.584	0.702	0.757	2.104	4.006	-	0.669
A-B	X60	2	0.958	3.226	-	-	0.642	1.145	4.013	-	-	0.615
A-B	X65	2	1.169	4.389	-	-	0.605	1.396	-	-	-	0.591
A-B	X70	2	1.274	4.994	-	-	0.622	1.493	-	-	-	0.608
A-C	X60	1	0.454	1.062	1.813	2.298	0.747	0.497	1.147	1.941	2.450	0.726
A-C	X65	1	0.497	1.219	2.141	2.748	0.737	0.547	1.322	2.307	2.951	0.714
A-C	X70	1	0.549	1.415	2.570	3.346	0.715	0.603	1.538	2.773	3.600	0.688
A-C	X60	2	0.761	2.338	4.731	-	0.655	0.852	2.635	-	-	0.625
A-C	X65	2	0.907	3.042	-	-	0.623	1.596	-	-	-	0.608
A-C	X70	2	1.049	3.813	-	-	0.598	1.170	4.302	-	-	0.583
B-C	X60	1	1.208	3.842	-	-	0.704	1.463	4.406	-	-	0.673
B-C	X65	1	1.395	4.796	-	-	0.677	1.640	-	-	-	0.663
B-C	X70	1	1.417	4.890	-	-	0.696	1.812	-	-	-	0.653
B-C	X60	2	2.519	-	-	-	0.590	2.751	-	-	-	0.611
B-C	X65	2	2.630	-	-	-	0.608	3.092	-	-	-	0.600
B-C	X70	2	2.863	-	-	-	0.593	2.940	-	-	-	0.616
			Cohesive soil deposit – Trench type TB					Cohesionless soil deposit – Trench type TB				
Soil	Steel Grade	h (m)	PGV _{m,OLS} (m/s)	PGV _{m,PILS} (m/s)	PGV _{m,ULS} (m/s)	PGV _{m,GLS} (m/s)	β_{tot}	PGV _{m,OLS} (m/s)	PGV _{m,PILS} (m/s)	PGV _{m,ULS} (m/s)	PGV _{m,GLS} (m/s)	β_{tot}
A-B	X60	1	0.267	0.480	0.694	0.818	0.997	0.303	0.542	0.782	0.920	0.992
A-B	X65	1	0.274	0.494	0.716	0.843	1.034	0.312	0.559	0.807	0.950	1.037
A-B	X70	1	0.288	0.530	0.778	0.923	1.039	0.331	0.610	0.897	1.063	1.037
A-B	X60	2	0.279	0.502	0.726	0.855	1.145	0.317	0.571	0.836	0.987	1.131
A-B	X65	2	0.296	0.546	0.803	0.952	1.158	0.343	0.617	0.964	1.150	1.130
A-B	X70	2	0.325	0.629	0.953	1.146	1.124	0.375	0.746	1.135	1.372	1.097
A-C	X60	1	0.246	0.432	0.615	0.719	1.012	0.269	0.464	0.655	0.763	0.998
A-C	X65	1	0.250	0.436	0.618	0.721	1.031	0.273	0.470	0.661	0.769	1.036
A-C	X70	1	0.262	0.465	0.666	0.782	1.046	0.286	0.501	0.713	0.834	1.051
A-C	X60	2	0.255	0.442	0.626	0.729	1.166	0.273	0.470	0.661	0.769	1.179
A-C	X65	2	0.268	0.472	0.675	0.790	1.206	0.289	0.508	0.726	0.850	1.206
A-C	X70	2	0.288	0.526	0.768	0.908	1.194	0.313	0.571	0.836	0.989	1.193
B-C	X60	1	0.498	1.111	1.844	2.308	0.905	0.614	1.453	2.501	3.180	0.841
B-C	X65	1	0.543	1.255	2.128	2.688	0.933	0.656	1.613	2.844	3.656	0.867
B-C	X70	1	0.601	1.464	2.566	3.289	0.919	0.743	1.954	3.595	4.708	0.841
B-C	X60	2	0.590	1.429	2.494	3.192	0.991	0.728	1.910	3.507	4.589	0.889
B-C	X65	2	0.678	1.766	3.228	4.216	0.960	0.866	2.491	4.849	-	0.840
B-C	X70	2	0.812	2.340	4.560	-	0.887	1.036	3.295	-	-	0.802

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1 **Table A4** Median peak ground velocity corresponding to the limit states, $PGV_{m,l}$, and total lognormal
2 standard deviation, β_{tot} , for 508.0 mm pipelines embedded in soil deposit of depth $H = 30$ m (- : the
3 limit state is not reached).
4

			Cohesive soil deposit – Trench type TA					Cohesionless soil deposit – Trench type TA				
Soil	Steel Grade	Burial depth, h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	1.139	3.356	-	-	0.791	0.802	2.336	4.524	-	0.758
A-B	X65	1	0.841	2.648	-	-	0.774	0.901	2.803	-	-	0.738
A-B	X70	1	0.962	3.306	-	-	0.734	1.031	3.503	-	-	0.700
A-B	X60	2	1.535	0.000	-	-	0.628	1.816	-	-	-	0.618
A-B	X65	2	1.895	0.000	-	-	0.599	2.243	-	-	-	0.593
A-B	X70	2	2.201	0.000	-	-	0.581	2.564	-	-	-	0.578
A-C	X60	1	0.752	2.333	4.697	-	0.773	0.798	2.530	-	-	0.761
A-C	X65	1	0.867	2.894	-	-	0.754	0.933	3.244	-	-	0.738
A-C	X70	1	1.091	3.713	-	-	0.714	1.065	4.045	-	-	0.700
A-C	X60	2	1.631	-	-	-	0.619	1.758	-	-	-	0.613
A-C	X65	2	2.044	-	-	-	0.592	2.192	-	-	-	0.589
A-C	X70	2	2.393	-	-	-	0.576	2.475	-	-	-	0.572
B-C	X60	1	2.214	-	-	-	0.711	2.328	-	-	-	0.687
B-C	X65	1	2.155	-	-	-	0.700	3.133	-	-	-	0.671
B-C	X70	1	2.410	-	-	-	0.681	3.537	-	-	-	0.646
B-C	X60	2	2.683	-	-	-	0.625	3.985	-	-	-	0.609
B-C	X65	2	3.670	-	-	-	0.607	4.552	-	-	-	0.597
B-C	X70	2	3.847	-	-	-	0.597	4.918	-	-	-	0.596
			Cohesive soil deposit – Trench type TB					Cohesionless soil deposit – Trench type TB				
Soil	Steel Grade	h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.328	0.639	0.967	1.319	1.051	0.356	0.693	1.046	1.426	1.019
A-B	X65	1	0.342	0.679	1.038	1.428	1.066	0.375	0.743	1.138	1.567	1.033
A-B	X70	1	0.363	0.750	1.174	1.643	1.053	0.398	0.825	1.295	1.818	1.021
A-B	X60	2	0.486	1.224	2.166	3.326	0.863	0.486	1.224	2.166	3.326	0.863
A-B	X65	2	0.562	1.551	2.906	4.658	0.857	0.742	2.443	-	0.000	0.781
A-B	X70	2	0.954	3.830	-	-	0.635	0.890	3.277	-	0.000	0.726
A-C	X60	1	0.314	0.627	0.960	1.322	1.027	0.329	0.667	1.034	1.437	1.017
A-C	X65	1	0.329	0.667	1.033	1.435	1.045	0.345	0.717	1.126	1.581	1.039
A-C	X70	1	0.350	0.740	1.176	1.665	1.035	0.369	0.800	1.290	1.848	1.026
A-C	X60	2	0.571	1.694	3.320	-	0.732	0.645	2.079	4.287	-	0.697
A-C	X65	2	0.724	2.523	-	-	0.684	0.832	3.178	-	-	0.654
A-C	X70	2	0.920	3.806	-	-	0.640	1.056	4.766	-	-	0.614
B-C	X60	1	0.847	2.480	4.817	-	0.917	0.898	2.714	-	-	0.854
B-C	X65	1	0.717	1.907	3.493	-	0.909	0.987	3.134	-	-	0.847
B-C	X70	1	0.919	2.820	-	-	0.899	1.075	3.596	-	-	0.826
B-C	X60	2	1.188	4.101	-	-	0.879	1.902	-	-	-	0.696
B-C	X65	2	1.375	-	-	-	0.851	2.431	-	-	-	0.660
B-C	X70	2	1.392	-	-	-	0.815	2.715	-	-	-	0.628

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1 **Table A5** Median peak ground velocity corresponding to the limit states, $PGV_{m,l}$, and total lognormal
2 standard deviation, β_{tot} , for 508.0 mm pipelines embedded in soil deposit of depth $H = 60$ m (- : the
3 limit state is not reached).
4

			Cohesive soil deposit – Trench type TA					Cohesionless soil deposit – Trench type TA				
Soil	Steel Grade	Burial depth, h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.491	1.226	2.158	3.301	0.860	0.548	1.407	2.521	3.907	0.844
A-B	X65	1	0.531	1.381	2.495	3.889	0.879	0.605	1.643	3.047	4.846	0.833
A-B	X70	1	0.962	3.306	-	-	0.734	0.658	1.883	3.606	0.000	0.846
A-B	X60	2	0.956	3.504	-	-	0.693	1.137	4.529	-	-	0.648
A-B	X65	2	1.133	4.594	-	-	0.668	1.283	-	-	-	0.672
A-B	X70	2	1.303	0.000	-	-	0.632	1.629	-	-	-	0.631
A-C	X60	1	0.445	1.044	1.770	2.631	0.864	0.470	1.120	1.915	2.867	0.873
A-C	X65	1	0.471	1.140	1.965	2.959	0.888	0.508	1.247	2.174	3.301	0.861
A-C	X70	1	0.575	1.311	2.338	3.610	0.877	0.554	1.450	2.630	4.113	0.875
A-C	X60	2	0.831	2.803	-	-	0.679	0.964	3.565	-	-	0.655
A-C	X65	2	1.001	3.781	-	-	0.651	1.063	4.150	-	-	0.681
A-C	X70	2	1.157	4.807	-	-	0.628	1.217	-	-	-	0.643
B-C	X60	1	1.013	3.221	-	-	0.802	1.061	3.358	-	-	0.790
B-C	X65	1	1.072	3.513	-	-	0.771	1.223	4.167	-	-	0.800
B-C	X70	1	1.096	3.622	-	-	0.772	1.312	4.655	-	-	0.764
B-C	X60	2	1.880	-	-	-	0.636	2.224	-	-	-	0.658
B-C	X65	2	2.211	-	-	-	0.619	2.690	-	-	-	0.624
B-C	X70	2	2.193	-	-	-	0.652	3.099	-	-	-	0.605
			Cohesive soil deposit – Trench type TB					Cohesionless soil deposit – Trench type TB				
Soil	Steel Grade	h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.260	0.493	0.733	0.987	0.976	0.275	0.513	0.753	1.005	0.987
A-B	X65	1	0.268	0.506	0.751	1.011	0.994	0.285	0.536	0.791	1.060	1.002
A-B	X70	1	0.281	0.548	0.827	1.126	0.981	0.302	0.587	0.885	1.205	0.984
A-B	X60	2	0.310	0.623	0.960	1.327	0.983	0.328	0.657	1.008	1.392	1.005
A-B	X65	2	0.339	0.714	1.133	1.602	0.985	0.368	0.789	1.264	1.801	1.011
A-B	X70	2	0.381	0.869	1.448	2.124	0.951	0.406	0.929	1.550	2.278	0.961
A-C	X60	1	0.244	0.441	0.636	0.838	0.988	0.258	0.475	0.693	0.921	0.985
A-C	X65	1	0.253	0.460	0.667	0.881	0.994	0.267	0.493	0.722	0.961	0.997
A-C	X70	1	0.264	0.490	0.719	0.959	0.980	0.278	0.527	0.782	1.052	0.977
A-C	X60	2	0.294	0.568	0.853	1.159	0.994	0.305	0.603	0.919	1.261	0.969
A-C	X65	2	0.320	0.649	1.006	1.398	0.968	0.340	0.719	1.142	1.617	0.930
A-C	X70	2	0.352	0.762	1.229	1.760	0.930	0.367	0.812	1.328	1.921	0.914
B-C	X60	1	0.456	1.027	1.699	2.478	1.012	0.508	1.185	2.001	2.966	1.008
B-C	X65	1	0.483	1.115	1.872	2.761	1.033	0.550	1.344	2.337	3.540	1.010
B-C	X70	1	0.522	1.269	2.198	3.322	1.006	0.599	1.551	2.794	4.346	0.973
B-C	X60	2	0.661	1.819	3.401	-	0.967	0.689	1.878	3.490	-	0.979
B-C	X65	2	0.682	1.880	3.521	-	0.984	0.789	2.307	4.481	-	0.947
B-C	X70	2	0.832	2.625	-	-	0.886	0.863	2.696	-	-	0.906

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1 **Table A6** Median peak ground velocity corresponding to the limit states, $PGV_{m,l}$, and total lognormal
2 standard deviation, β_{tot} , for 508.0 mm pipelines embedded in soil deposit of depth $H = 120$ m (- : the
3 limit state is not reached).
4

			Cohesive soil deposit – Trench type TA					Cohesionless soil deposit – Trench type TA				
Soil	Steel Grade	Burial depth, h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.380	0.904	1.542	2.305	0.893	0.437	1.043	1.786	2.676	0.863
A-B	X65	1	0.405	0.989	1.717	2.599	0.901	0.470	1.163	2.037	3.102	0.861
A-B	X70	1	0.445	1.151	2.072	3.222	0.876	0.518	1.364	2.481	3.889	0.827
A-B	X60	2	0.737	2.566	-	-	0.663	0.860	3.087	-	-	0.646
A-B	X65	2	0.892	3.493	-	-	0.635	1.032	4.142	-	-	0.623
A-B	X70	2	1.044	4.521	-	-	0.609	1.178	-	-	-	0.611
A-C	X60	1	0.290	0.569	0.863	1.181	0.997	0.361	0.785	1.269	1.821	0.927
A-C	X65	1	0.307	0.602	0.922	1.268	1.028	0.380	0.841	1.376	1.991	0.936
A-C	X70	1	0.320	0.654	1.017	1.419	1.024	0.400	0.855	1.370	1.951	0.973
A-C	X60	2	0.600	1.864	3.754	-	0.698	0.623	1.876	3.707	-	0.723
A-C	X65	2	0.697	2.267	-	-	0.668	0.739	2.469	-	-	0.681
A-C	X70	2	0.778	2.819	-	-	0.662	0.881	3.315	-	-	0.640
B-C	X60	1	0.815	2.600	-	-	0.759	0.858	2.718	-	-	0.755
B-C	X65	1	0.890	3.070	-	-	0.746	1.030	3.631	-	-	0.739
B-C	X70	1	0.944	3.351	-	-	0.736	1.071	3.840	-	-	0.749
B-C	X60	2	2.028	-	-	-	0.595	2.063	-	-	-	0.610
B-C	X65	2	2.211	-	-	-	0.619	2.037	-	-	-	0.635
B-C	X70	2	2.037	-	-	-	0.619	2.352	-	-	-	0.609
			Cohesive soil deposit – Trench type TB					Cohesionless soil deposit – Trench type TB				
Soil	Steel Grade	h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.201	0.363	0.523	0.687	1.081	0.220	0.407	0.581	0.760	1.076
A-B	X65	1	0.210	0.380	0.549	0.723	1.117	0.235	0.419	0.598	0.781	1.117
A-B	X70	1	0.220	0.410	0.601	0.802	1.107	0.246	0.449	0.650	0.858	1.098
A-B	X60	2	0.237	0.443	0.653	0.873	1.129	0.267	0.493	0.733	0.980	1.125
A-B	X65	2	0.255	0.491	0.737	1.001	1.130	0.289	0.561	0.845	1.151	1.135
A-B	X70	2	0.273	0.550	0.848	1.173	1.115	0.309	0.627	0.972	1.350	1.087
A-C	X60	1	0.188	0.328	0.463	0.599	1.087	0.206	0.352	0.491	0.629	1.085
A-C	X65	1	0.193	0.335	0.471	0.608	1.113	0.212	0.361	0.503	0.645	1.117
A-C	X70	1	0.205	0.368	0.528	0.694	1.096	0.223	0.395	0.561	0.731	1.106
A-C	X60	2	0.227	0.421	0.617	0.822	1.119	0.237	0.425	0.610	0.800	1.160
A-C	X65	2	0.230	0.420	0.610	0.807	1.188	0.250	0.457	0.665	0.880	1.190
A-C	X70	2	0.259	0.514	0.785	1.080	1.114	0.273	0.530	0.798	1.086	1.147
B-C	X60	1	0.337	0.729	1.175	1.681	1.012	0.390	0.877	1.448	2.110	0.948
B-C	X65	1	0.350	0.769	1.252	1.804	1.025	0.417	0.969	1.633	2.418	0.958
B-C	X70	1	0.373	0.853	1.422	2.088	1.012	0.450	1.100	1.912	2.897	0.940
B-C	X60	2	0.431	1.037	1.784	2.682	1.029	0.527	1.378	2.498	3.907	0.943
B-C	X65	2	0.487	1.261	2.271	3.534	1.007	0.690	2.019	3.922	-	0.868
B-C	X70	2	0.585	1.700	3.289	-	0.925	0.717	2.300	4.727	-	0.814

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1 **Table A7** Median peak ground velocity corresponding to the limit states, $PGV_{m,l}$, and total lognormal
2 standard deviation, β_{tot} , for 762.0 mm pipelines embedded in soil deposit of depth $H = 30$ m (- : the
3 limit state is not reached).
4

			Cohesive soil deposit – Trench type TA					Cohesionless soil deposit – Trench type TA				
Soil	Steel Grade	Burial depth, h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	1.230	3.962	-	-	0.767	1.407	4.789	-	-	0.754
A-B	X65	1	1.321	4.391	-	-	0.772	1.521	-	-	-	0.747
A-B	X70	1	1.656	-	-	-	0.690	1.736	-	-	-	0.708
A-B	X60	2	2.013	-	-	-	0.648	2.333	-	-	-	0.644
A-B	X65	2	2.321	-	-	-	0.637	2.636	-	-	-	0.629
A-B	X70	2	2.685	-	-	-	0.603	3.159	-	-	-	0.597
A-C	X60	1	1.343	4.740	-	-	0.732	1.407	4.789	-	-	0.754
A-C	X65	1	1.477	-	-	-	0.739	1.521	-	-	-	0.747
A-C	X70	1	1.696	-	-	-	0.700	1.736	-	-	-	0.708
A-C	X60	2	2.231	-	-	-	0.637	2.394	-	-	-	0.629
A-C	X65	2	2.561	-	-	-	0.625	2.733	-	-	-	0.616
A-C	X70	2	3.108	-	-	-	0.590	3.159	-	-	-	0.597
B-C	X60	1	-	-	-	-	-	-	-	-	-	-
B-C	X65	1	-	-	-	-	-	-	-	-	-	-
B-C	X70	1	-	-	-	-	-	-	-	-	-	-
B-C	X60	2	-	-	-	-	-	-	-	-	-	-
B-C	X65	2	-	-	-	-	-	-	-	-	-	-
B-C	X70	2	-	-	-	-	-	-	-	-	-	-
			Cohesive soil deposit – Trench type TB					Cohesionless soil deposit – Trench type TB				
Soil	Steel Grade	h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.649	1.711	3.118	4.609	0.989	0.574	1.350	2.291	3.233	1.030
A-B	X65	1	0.557	1.297	2.188	3.075	1.079	0.623	1.552	2.715	3.908	1.034
A-B	X70	1	0.612	1.507	2.632	3.784	1.052	0.696	1.828	3.321	4.900	0.993
A-B	X60	2	0.515	1.295	2.293	3.326	1.067	0.664	1.741	3.169	4.677	1.010
A-B	X65	2	0.648	1.683	3.035	4.457	1.059	0.783	2.275	4.400	-	0.986
A-B	X70	2	0.722	2.005	3.770	-	1.013	0.855	2.625	-	-	0.927
A-C	X60	1	0.513	1.195	2.015	2.834	1.015	0.529	1.238	2.095	2.951	1.020
A-C	X65	1	0.566	1.393	2.432	3.496	1.030	0.597	1.502	2.658	3.854	1.005
A-C	X70	1	0.616	1.591	2.862	4.195	1.021	0.649	1.711	3.118	4.609	0.989
A-C	X60	2	0.618	1.627	2.962	4.377	1.022	0.685	1.697	2.973	4.284	1.032
A-C	X65	2	0.685	1.932	3.670	-	1.011	0.736	2.172	4.241	-	0.968
A-C	X70	2	0.786	2.425	4.867	-	0.947	0.883	2.929	-	-	0.897
B-C	X60	1	1.484	-	-	-	0.818	2.008	-	-	-	0.751
B-C	X65	1	1.831	-	-	-	0.806	2.270	-	-	-	0.734
B-C	X70	1	2.039	-	-	-	0.784	2.611	-	-	-	0.701
B-C	X60	2	1.922	-	-	-	0.804	2.680	-	-	-	0.697
B-C	X65	2	2.505	-	-	-	0.747	3.004	-	-	-	0.670
B-C	X70	2	2.375	-	-	-	0.747	3.744	-	-	-	0.628

5

1 **Table A8.** Median peak ground velocity corresponding to the limit states, $PGV_{m,i}$, and total lognormal
2 standard deviation, β_{tot} , for 762.0 mm pipelines embedded in soil deposit of depth $H = 60$ m (- : the
3 limit state is not reached).
4

			Cohesive soil deposit – Trench type TA					Cohesionless soil deposit – Trench type TA				
Soil	Steel Grade	Burial depth, h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.921	2.827	-	-	0.772	1.034	3.259	-	-	0.778
A-B	X65	1	1.101	3.732	-	-	0.741	1.180	3.988	-	-	0.755
A-B	X70	1	1.656	-	-	-	0.712	1.431	-	-	-	0.719
A-B	X60	2	1.474	-	-	-	0.649	1.824	-	-	-	0.629
A-B	X65	2	1.890	-	-	-	0.611	2.331	-	-	-	0.602
A-B	X70	2	2.235	-	-	-	0.593	2.403	-	-	-	0.610
A-C	X60	1	0.774	2.158	4.069	-	0.795	0.832	2.370	4.530	-	0.792
A-C	X65	1	0.887	2.675	-	-	0.771	0.985	3.089	-	-	0.760
A-C	X70	1	1.056	3.568	-	-	0.712	1.160	4.005	-	-	0.717
A-C	X60	2	1.301	-	-	-	0.645	1.324	-	-	-	0.667
A-C	X65	2	1.470	-	-	-	0.642	1.683	-	-	-	0.626
A-C	X70	2	1.831	-	-	-	0.598	1.970	-	-	-	0.608
B-C	X60	1	1.995	-	-	-	0.720	2.071	-	-	-	0.724
B-C	X65	1	1.873	-	-	-	0.735	2.664	-	-	-	0.702
B-C	X70	1	2.280	-	-	-	0.684	2.887	-	-	-	0.673
B-C	X60	2	3.177	-	-	-	0.615	4.127	-	-	-	0.611
B-C	X65	2	3.552	-	-	-	0.603	4.956	-	-	-	0.592
B-C	X70	2	4.083	-	-	-	0.588	-	-	-	-	-
			Cohesive soil deposit – Trench type TB					Cohesionless soil deposit – Trench type TB				
Soil	Steel Grade	h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.375	0.763	1.183	1.575	1.035	0.431	0.922	1.477	2.007	0.982
A-B	X65	1	0.404	0.854	1.357	1.835	1.048	0.464	1.021	1.668	2.296	1.019
A-B	X70	1	0.424	0.919	1.482	2.024	1.093	0.495	1.154	1.940	2.721	1.042
A-B	X60	2	0.413	0.897	1.449	1.980	1.084	0.472	1.083	1.826	2.559	1.029
A-B	X65	2	0.477	1.131	1.929	2.731	1.032	0.596	1.666	2.963	4.416	0.935
A-B	X70	2	0.525	1.321	2.338	3.392	1.057	0.652	1.855	3.534	-	0.926
A-C	X60	1	0.338	0.645	0.961	1.246	1.052	0.367	0.725	1.104	1.452	1.008
A-C	X65	1	0.359	0.703	1.064	1.395	1.076	0.389	0.786	1.215	1.614	1.044
A-C	X70	1	0.383	0.778	1.206	1.604	1.089	0.419	0.886	1.406	1.901	1.061
A-C	X60	2	0.380	0.786	1.233	1.653	1.094	0.395	0.830	1.315	1.774	1.071
A-C	X65	2	0.403	0.855	1.362	1.843	1.119	0.456	1.049	1.754	2.453	1.045
A-C	X70	2	0.428	0.941	1.530	2.101	1.141	0.524	1.320	2.338	3.393	1.012
B-C	X60	1	0.791	2.125	3.916	-	0.946	0.866	2.370	4.417	-	0.953
B-C	X65	1	0.874	2.480	4.727	-	0.923	0.892	2.458	4.602	-	0.975
B-C	X70	1	0.961	2.879	-	-	0.893	0.910	2.526	4.750	-	0.971
B-C	X60	2	0.888	2.589	-	-	0.950	0.988	2.985	-	-	0.916
B-C	X65	2	1.074	3.513	-	-	0.886	1.181	3.934	-	-	0.871
B-C	X70	2	1.239	4.413	-	-	0.854	1.121	3.614	-	-	0.914

5

1 **Table A9.** Median peak ground velocity corresponding to the limit states, $PGV_{m,i}$, and total lognormal
2 standard deviation, β_{tot} , for 762.0 mm pipelines embedded in soil deposit of depth $H = 120$ m (- : the
3 limit state is not reached).
4

			Cohesive soil deposit – Trench type TA					Cohesionless soil deposit – Trench type TA				
Soil	Steel Grade	Burial depth, h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.715	2.099	4.086	-	0.746	0.835	2.491	4.898	-	0.703
A-B	X65	1	0.858	2.788	-	-	0.706	0.938	2.962	-	-	0.702
A-B	X70	1	1.018	3.659	-	-	0.669	1.173	4.253	-	-	0.635
A-B	X60	2	1.157	4.600	-	-	0.636	1.303	-	-	-	0.637
A-B	X65	2	1.431	-	-	-	0.612	1.685	-	-	-	0.603
A-B	X70	2	2.235	-	-	-	0.593	1.933	-	-	-	0.595
A-C	X60	1	0.572	1.499	2.722	4.014	0.799	0.623	1.620	2.926	4.301	0.778
A-C	X65	1	0.655	1.852	3.520	-	0.777	0.717	2.019	3.828	-	0.749
A-C	X70	1	0.770	2.393	4.824	-	0.726	0.833	2.560	-	-	0.691
A-C	X60	2	0.891	3.139	-	-	0.659	0.995	3.523	-	-	0.644
A-C	X65	2	1.121	4.426	-	-	0.624	1.257	-	-	-	0.608
A-C	X70	2	1.370	-	-	-	0.595	1.461	-	-	-	0.597
B-C	X60	1	1.596	-	-	-	0.714	1.674	-	-	-	0.703
B-C	X65	1	1.628	-	-	-	0.720	1.963	-	-	-	0.687
B-C	X70	1	2.020	-	-	-	0.668	2.214	-	-	-	0.669
B-C	X60	2	3.177	-	-	-	0.615	3.119	-	-	-	0.609
B-C	X65	2	3.552	-	-	-	0.603	3.728	-	-	-	0.577
B-C	X70	2	4.083	-	-	-	0.588	4.199	-	-	-	0.575
			Cohesive soil deposit – Trench type TB					Cohesionless soil deposit – Trench type TB				
Soil	Steel Grade	h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.289	0.560	0.842	1.098	1.092	0.334	0.649	0.979	1.280	1.078
A-B	X65	1	0.305	0.603	0.919	1.209	1.115	0.357	0.704	1.078	1.424	1.095
A-B	X70	1	0.329	0.676	1.055	1.410	1.105	0.378	0.782	1.226	1.643	1.079
A-B	X60	2	0.308	0.615	0.943	1.246	1.158	0.359	0.741	1.160	1.554	1.105
A-B	X65	2	0.339	0.714	1.132	1.528	1.146	0.402	0.820	1.454	2.003	1.088
A-B	X70	2	0.374	0.837	1.376	1.903	1.120	0.448	1.060	1.804	2.552	1.029
A-C	X60	1	0.259	0.474	0.689	0.879	1.143	0.283	0.511	0.736	0.934	1.140
A-C	X65	1	0.272	0.507	0.745	0.958	1.169	0.297	0.546	0.796	1.017	1.180
A-C	X70	1	0.288	0.549	0.819	1.064	1.179	0.311	0.581	0.856	1.102	1.194
A-C	X60	2	0.266	0.486	0.706	0.900	1.267	0.295	0.552	0.814	1.048	1.233
A-C	X65	2	0.303	0.600	0.915	1.206	1.232	0.334	0.673	1.039	1.379	1.207
A-C	X70	2	0.320	0.652	1.014	1.352	1.227	0.349	0.723	1.134	1.521	1.209
B-C	X60	1	0.620	1.670	3.082	4.595	0.912	0.714	1.971	3.695	-	0.887
B-C	X65	1	0.683	1.951	3.734	-	0.900	0.769	2.200	4.213	-	0.881
B-C	X70	1	0.781	2.419	4.866	-	0.857	0.813	2.384	4.639	-	0.883
B-C	X60	2	0.749	2.311	4.638	-	0.864	0.817	2.489	4.957	-	0.879
B-C	X65	2	0.887	3.023	-	-	0.840	1.174	4.521	-	-	0.786
B-C	X70	2	1.088	4.200	-	-	0.783	1.337	-	-	-	0.741

5

1 **Table A10.** Median peak ground velocity corresponding to the limit states, $PGV_{m,l}$, and total lognormal
2 standard deviation, β_{tot} , for 914.4 mm pipelines embedded in soil deposit of depth $H = 30$ m (- : the
3 limit state is not reached).
4

			Cohesive soil deposit – Trench type TA					Cohesionless soil deposit – Trench type TA				
Soil	Steel Grade	Burial depth, h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.736	2.160	4.205	-	0.879	0.826	2.563	-	-	0.850
A-B	X65	1	0.922	3.101	-	-	0.807	1.042	3.716	-	-	0.779
A-B	X70	1	1.081	4.021	-	-	0.761	1.228	4.858	-	-	0.735
A-B	X60	2	0.945	3.426	-	-	0.816	1.009	3.729	-	-	0.833
A-B	X65	2	1.286	-	-	-	0.720	2.073	-	-	-	0.556
A-B	X70	2	1.336	-	-	-	0.733	1.536	-	-	-	0.723
A-C	X60	1	0.763	2.411	4.912	-	0.861	0.796	2.546	-	-	0.836
A-C	X65	1	1.009	-	-	-	0.755	1.011	3.740	-	-	0.763
A-C	X70	1	1.079	4.184	-	-	0.768	1.323	-	-	-	0.755
A-C	X60	2	0.927	3.460	-	-	0.840	1.958	-	-	-	0.541
A-C	X65	2	1.832	-	-	-	0.625	1.268	-	-	-	0.745
A-C	X70	2	1.437	-	-	-	0.723	1.519	-	-	-	0.717
B-C	X60	1	3.185	-	-	-	0.545	2.905	-	-	-	0.736
B-C	X65	1	2.974	-	-	-	0.702	4.900	-	-	-	0.697
B-C	X70	1	3.646	-	-	-	0.678	-	-	-	-	0.669
B-C	X60	2	2.819	-	-	-	0.705	-	-	-	-	0.688
B-C	X65	2	4.159	-	-	-	0.655	-	-	-	-	0.646
B-C	X70	2	-	-	-	-	-	-	-	-	-	-
			Cohesive soil deposit – Trench type TB					Cohesionless soil deposit – Trench type TB				
Soil	Steel Grade	h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.351	0.738	1.169	1.836	1.148	0.375	0.813	1.310	2.091	1.173
A-B	X65	1	0.401	0.900	1.484	2.424	1.101	0.427	0.983	1.646	2.728	1.110
A-B	X70	1	0.440	1.045	1.783	3.011	1.090	0.488	1.193	2.095	3.637	1.056
A-B	X60	2	0.357	0.769	1.234	1.963	1.233	0.391	0.821	1.491	2.461	1.200
A-B	X65	2	0.409	0.946	1.589	2.641	1.166	0.427	1.064	1.702	2.856	1.178
A-B	X70	2	0.614	1.954	4.000	-	0.870	0.587	1.758	3.463	-	0.951
A-C	X60	1	0.333	0.709	1.131	1.787	1.155	0.337	0.706	1.115	1.745	1.198
A-C	X65	1	0.374	0.836	1.373	2.235	1.138	0.383	0.852	1.397	2.269	1.144
A-C	X70	1	0.432	1.067	1.866	3.229	1.066	0.451	1.134	2.007	3.511	1.040
A-C	X60	2	0.347	0.773	1.269	2.063	1.221	0.347	0.773	1.269	2.063	1.221
A-C	X65	2	0.384	0.885	1.483	2.460	1.210	0.392	0.911	1.536	2.563	1.197
A-C	X70	2	0.546	1.669	3.329	-	0.950	0.553	1.693	3.383	-	0.938
B-C	X60	1	0.852	2.614	-	-	0.955	0.858	2.628	-	-	0.916
B-C	X65	1	0.927	2.959	-	-	0.894	1.200	4.450	-	-	0.838
B-C	X70	1	1.131	4.130	-	-	0.823	1.475	-	-	-	0.769
B-C	X60	2	0.993	3.549	-	-	0.877	1.067	3.870	-	-	0.888
B-C	X65	2	0.962	3.222	-	-	0.907	1.385	-	-	-	0.798
B-C	X70	2	1.571	-	-	-	0.710	1.884	-	-	-	0.708

5

1 **Table A11.** Median peak ground velocity corresponding to the limit states, $PGV_{m,l}$, and total lognormal
2 standard deviation, β_{tot} , for 914.4 mm pipelines embedded in soil deposit of depth $H = 60$ m (- : the
3 limit state is not reached).
4

			Cohesive soil deposit – Trench type TA					Cohesionless soil deposit – Trench type TA				
Soil	Steel Grade	Burial depth, h (m)	PGV _{m,OLS} (m/s)	PGV _{m,PILS} (m/s)	PGV _{m,ULS} (m/s)	PGV _{m,GLS} (m/s)	β_{tot}	PGV _{m,OLS} (m/s)	PGV _{m,PILS} (m/s)	PGV _{m,ULS} (m/s)	PGV _{m,GLS} (m/s)	β_{tot}
A-B	X60	1	0.742	2.112	4.032	-	0.880	0.661	2.004	3.981	-	0.850
A-B	X65	1	0.695	2.216	4.539	-	0.813	0.794	2.657	-	-	0.791
A-B	X70	1	0.806	2.818	-	-	0.769	0.892	3.183	-	-	0.770
A-B	X60	2	0.676	2.229	4.664	-	0.858	0.776	2.736	-	-	0.833
A-B	X65	2	0.882	3.387	-	-	0.770	1.050	4.422	-	-	0.743
A-B	X70	2	1.130	-	-	-	0.688	1.184	-	-	-	0.738
A-C	X60	1	0.508	1.342	2.447	4.410	0.881	0.542	1.463	2.705	4.938	0.880
A-C	X65	1	0.601	1.807	3.523	-	0.821	0.652	1.954	3.854	-	0.830
A-C	X70	1	0.693	2.198	4.489	-	0.785	0.765	2.545	-	-	0.784
A-C	X60	2	0.600	1.828	3.641	-	0.865	0.645	2.048	4.186	-	0.866
A-C	X65	2	0.767	2.688	-	-	0.780	0.837	3.104	-	-	0.784
A-C	X70	2	0.965	2.951	-	-	0.692	1.028	4.363	-	-	0.705
B-C	X60	1	1.294	-	-	-	0.788	1.226	4.432	-	-	0.808
B-C	X65	1	1.284	4.833	-	-	0.773	1.797	-	-	-	0.759
B-C	X70	1	1.440	-	-	-	0.741	2.143	-	-	-	0.700
B-C	X60	2	1.264	4.968	-	-	0.767	1.791	-	-	-	0.741
B-C	X65	2	1.541	-	-	-	0.721	2.461	-	-	-	0.675
B-C	X70	2	2.094	-	-	-	0.651	2.962	-	-	-	0.646
			Cohesive soil deposit – Trench type TB					Cohesionless soil deposit – Trench type TB				
Soil	Steel Grade	h (m)	PGV _{m,OLS} (m/s)	PGV _{m,PILS} (m/s)	PGV _{m,ULS} (m/s)	PGV _{m,GLS} (m/s)	β_{tot}	PGV _{m,OLS} (m/s)	PGV _{m,PILS} (m/s)	PGV _{m,ULS} (m/s)	PGV _{m,GLS} (m/s)	β_{tot}
A-B	X60	1	0.268	0.523	0.790	1.185	1.103	0.290	0.569	0.862	1.296	1.101
A-B	X65	1	0.296	0.595	0.917	1.400	1.083	0.325	0.680	1.068	1.663	1.048
A-B	X70	1	0.333	0.728	1.180	1.895	1.015	0.378	0.871	1.459	2.418	0.968
A-B	X60	2	0.278	0.567	0.879	1.353	1.154	0.313	0.683	1.105	1.772	1.093
A-B	X65	2	0.302	0.621	0.971	1.503	1.162	0.330	0.765	1.126	1.782	1.134
A-B	X70	2	0.409	1.073	1.947	3.494	0.927	0.465	1.158	2.035	3.535	0.986
A-C	X60	1	0.255	0.482	0.715	1.051	1.060	0.267	0.510	0.759	1.123	1.072
A-C	X65	1	0.281	0.550	0.833	1.252	1.032	0.289	0.560	0.842	1.257	1.074
A-C	X70	1	0.300	0.602	0.926	1.412	1.064	0.324	0.676	1.066	1.665	1.011
A-C	X60	2	0.257	0.488	0.726	1.071	1.156	0.271	0.538	0.821	1.244	1.127
A-C	X65	2	0.286	0.569	0.871	1.321	1.121	0.294	0.589	0.905	1.379	1.148
A-C	X70	2	0.378	0.943	1.660	2.887	0.882	0.376	0.920	1.599	2.749	0.931
B-C	X60	1	0.477	1.138	1.948	3.299	1.111	0.520	1.284	2.244	3.878	1.056
B-C	X65	1	0.556	1.408	2.500	4.389	1.044	0.636	1.756	3.289	-	0.976
B-C	X70	1	0.665	1.960	3.825	-	0.924	0.767	2.403	4.872	-	0.909
B-C	X60	2	0.538	1.445	2.664	4.852	1.056	0.584	1.605	2.998	-	1.059
B-C	X65	2	0.549	1.427	2.576	4.595	1.082	0.625	1.729	3.242	-	1.039
B-C	X70	2	0.885	3.293	-	-	0.828	1.019	4.060	-	-	0.793

5

1 **Table A12.** Median peak ground velocity corresponding to the limit states, $PGV_{m,l}$, and total lognormal
2 standard deviation, β_{tot} , for 914.4 mm pipelines embedded in soil deposit of depth $H = 120$ m (- : the
3 limit state is not reached).
4

			Cohesive soil deposit – Trench type TA					Cohesionless soil deposit – Trench type TA				
Soil	Steel Grade	Burial depth, h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.428	1.130	2.061	3.713	0.889	0.508	1.382	2.567	4.708	0.852
A-B	X65	1	0.517	1.518	2.953	-	0.824	0.508	1.382	2.567	4.708	0.839
A-B	X70	1	0.615	2.013	4.191	-	0.768	0.710	2.350	4.927	-	0.731
A-B	X60	2	0.518	1.604	3.229	-	0.863	0.611	1.979	4.091	-	0.827
A-B	X65	2	0.706	2.642	-	-	0.741	0.611	1.979	4.091	-	0.827
A-B	X70	2	0.835	3.447	-	-	0.714	0.968	4.122	-	-	0.702
A-C	X60	1	0.366	0.881	1.517	2.582	0.920	0.405	0.978	1.688	2.881	0.903
A-C	X65	1	0.517	1.807	3.523	-	0.821	0.652	1.954	3.854	-	0.830
A-C	X70	1	0.615	1.413	2.711	-	0.815	0.547	1.576	3.033	-	0.790
A-C	X60	2	0.418	1.124	2.072	3.773	0.917	0.468	1.298	2.438	4.522	0.889
A-C	X65	2	0.767	2.688	-	-	0.780	0.470	1.311	2.473	4.606	0.890
A-C	X70	2	0.687	2.534	-	-	0.714	0.764	2.897	-	-	0.699
B-C	X60	1	0.934	3.468	-	-	0.799	1.005	3.680	-	-	0.787
B-C	X65	1	1.104	4.455	-	-	0.753	1.318	-	-	-	0.736
B-C	X70	1	1.267	-	-	-	0.717	1.552	-	-	-	0.709
B-C	X60	2	1.090	4.631	-	-	0.777	1.275	-	-	-	0.774
B-C	X65	2	1.380	-	-	-	0.720	1.731	-	-	-	0.656
B-C	X70	2	1.713	-	-	-	0.695	2.139	-	-	-	0.662
			Cohesive soil deposit – Trench type TB					Cohesionless soil deposit – Trench type TB				
Soil	Steel Grade	h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.206	0.374	0.540	0.775	1.223	0.235	0.424	0.609	0.870	1.229
A-B	X65	1	0.225	0.419	0.616	0.899	1.194	0.255	0.481	0.708	1.032	1.198
A-B	X70	1	0.252	0.504	0.775	1.181	1.132	0.287	0.576	0.885	1.348	1.124
A-B	X60	2	0.218	0.417	0.624	0.926	1.268	0.242	0.458	0.680	1.000	1.267
A-B	X65	2	0.225	0.417	0.609	0.884	1.297	0.256	0.461	0.709	1.037	1.268
A-B	X70	2	0.310	0.747	1.286	2.190	1.039	0.335	0.784	1.327	2.222	1.023
A-C	X60	1	0.193	0.340	0.482	0.678	1.245	0.212	0.369	0.520	0.728	1.245
A-C	X65	1	0.210	0.379	0.546	0.781	1.224	0.231	0.416	0.599	0.855	1.210
A-C	X70	1	0.236	0.460	0.694	1.039	1.150	0.256	0.486	0.724	1.069	1.158
A-C	X60	2	0.209	0.404	0.608	0.906	1.214	0.223	0.421	0.624	0.916	1.245
A-C	X65	2	0.211	0.378	0.543	0.774	1.333	0.228	0.409	0.587	0.837	1.328
A-C	X70	2	0.279	0.627	1.036	1.693	1.065	0.305	0.697	1.162	1.917	1.015
B-C	X60	1	0.353	0.813	1.360	2.254	1.111	0.428	1.072	1.891	3.297	1.008
B-C	X65	1	0.418	1.053	1.865	3.265	1.034	0.519	1.447	2.727	-	0.927
B-C	X70	1	0.494	1.397	2.657	4.987	0.972	0.616	1.939	3.938	-	0.866
B-C	X60	2	0.371	0.886	1.519	2.576	1.168	0.452	1.204	2.207	3.994	1.063
B-C	X65	2	0.434	1.134	2.054	3.676	1.087	0.540	1.581	3.070	-	0.956
B-C	X70	2	0.636	2.209	4.773	-	0.866	0.757	2.864	-	-	0.788

5

1 **Table A13** Median peak ground velocity corresponding to the limit states, $PGV_{m,l}$, and total lognormal
2 standard deviation, β_{tot} , for 1066.4 mm pipelines embedded in soil deposit of depth $H = 30$ m (- : the
3 limit state is not reached).
4

			Cohesive soil deposit – Trench type TA					Cohesionless soil deposit – Trench type TA				
Soil	Steel Grade	Burial depth, h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.995	3.221	-	-	0.819	1.135	3.905	-	-	0.801
A-B	X65	1	1.204	4.340	-	-	0.770	1.400	-	-	-	0.751
A-B	X70	1	1.366	-	-	-	0.735	1.502	-	-	-	0.734
A-B	X60	2	1.495	-	-	-	0.715	1.748	-	-	-	0.704
A-B	X65	2	2.000	-	-	-	0.658	2.333	-	-	-	0.650
A-B	X70	2	2.517	-	-	-	0.548	2.772	-	-	-	0.624
A-C	X60	1	1.063	3.761	-	-	0.793	1.108	3.958	-	-	0.786
A-C	X65	1	1.321	-	-	-	0.741	1.355	-	-	-	0.742
A-C	X70	1	1.457	-	-	-	0.727	1.508	-	-	-	0.721
A-C	X60	2	1.600	-	-	-	0.705	1.709	-	-	-	0.680
A-C	X65	2	2.128	-	-	-	0.651	2.293	-	-	-	0.631
A-C	X70	2	2.692	-	-	-	0.617	2.862	-	-	-	0.613
B-C	X60	1	4.547	-	-	-	0.700	4.427	-	-	-	0.704
B-C	X65	1	4.592	-	-	-	0.674	-	-	-	-	-
B-C	X70	1	-	-	-	-	0.661	-	-	-	-	-
B-C	X60	2	4.920	-	-	-	0.663	-	-	-	-	-
B-C	X65	2	-	-	-	-	-	-	-	-	-	-
B-C	X70	2	-	-	-	-	-	-	-	-	-	-
			Cohesive soil deposit – Trench type TB					Cohesionless soil deposit – Trench type TB				
Soil	Steel Grade	h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.421	0.917	1.484	2.291	1.139	0.452	1.013	1.667	2.615	1.128
A-B	X65	1	0.457	1.027	1.695	2.666	1.151	0.507	1.204	2.057	3.336	1.096
A-B	X70	1	0.543	1.382	2.462	4.147	1.027	0.625	1.717	3.214	-	0.942
A-B	X60	2	0.448	1.025	1.711	2.717	1.152	0.507	1.251	2.208	3.668	1.083
A-B	X65	2	0.587	1.615	3.020	-	0.998	0.589	1.571	2.941	-	1.039
A-B	X70	2	0.682	2.043	4.027	-	0.960	0.810	2.706	-	-	0.871
A-C	X60	1	0.410	0.920	1.515	2.378	1.125	0.428	0.975	1.622	2.569	1.095
A-C	X65	1	0.461	1.100	1.884	3.061	1.101	0.482	1.171	2.027	3.327	1.068
A-C	X70	1	0.540	1.440	2.641	4.566	1.005	0.570	1.557	2.899	-	0.962
A-C	X60	2	0.456	1.116	1.943	3.204	1.100	0.480	1.218	2.166	3.644	1.048
A-C	X65	2	0.538	1.453	2.687	4.679	1.058	0.566	1.583	2.989	-	1.014
A-C	X70	2	0.733	2.446	-	-	0.894	0.744	2.492	-	-	0.900
B-C	X60	1	1.200	4.137	-	-	0.877	1.264	4.476	-	-	0.812
B-C	X65	1	1.253	4.409	-	-	0.843	1.773	-	-	-	0.771
B-C	X70	1	1.428	-	-	-	0.823	2.105	-	-	-	0.720
B-C	X60	2	1.145	3.920	-	-	0.876	1.798	-	-	-	0.750
B-C	X65	2	1.603	-	-	-	0.786	2.303	-	-	-	0.703
B-C	X70	2	1.956	-	-	-	0.769	3.092	-	-	-	0.673

5

1 **Table A14** Median peak ground velocity corresponding to the limit states, $PGV_{m,l}$, and total lognormal
2 standard deviation, β_{tot} , for 1066.4 mm pipelines embedded in soil deposit of depth $H = 60$ m (- : the
3 limit state is not reached).
4

			Cohesive soil deposit – Trench type TA					Cohesionless soil deposit – Trench type TA				
Soil	Steel Grade	Burial depth, h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.772	2.437	4.961	-	0.809	0.807	2.482	4.972	-	0.836
A-B	X65	1	0.938	3.295	-	-	0.762	0.979	3.357	-	-	0.793
A-B	X70	1	1.144	4.558	-	-	0.702	1.342	-	-	-	0.697
A-B	X60	2	1.059	4.189	-	-	0.736	1.239	-	-	-	0.722
A-B	X65	2	1.373	-	-	-	0.677	1.694	-	-	-	0.661
A-B	X70	2	1.720	-	-	-	0.633	2.053	-	-	-	0.629
A-C	X60	1	0.679	1.990	3.870	-	0.813	0.714	2.110	4.123	-	0.827
A-C	X65	1	0.817	2.662	-	-	0.760	0.867	2.857	-	-	0.768
A-C	X70	1	0.994	3.652	-	-	0.706	1.016	3.693	-	-	0.719
A-C	X60	2	0.900	3.221	-	-	0.747	0.991	3.763	-	-	0.748
A-C	X65	2	1.163	4.827	-	-	0.684	1.278	-	-	-	0.676
A-C	X70	2	1.396	-	-	-	0.645	1.538	-	-	-	0.647
B-C	X60	1	1.666	-	-	-	0.758	1.686	-	-	-	0.765
B-C	X65	1	1.770	-	-	-	0.720	2.471	-	-	-	0.725
B-C	X70	1	2.114	-	-	-	0.683	3.021	-	-	-	0.665
B-C	X60	2	2.362	-	-	-	0.682	2.749	-	-	-	0.688
B-C	X65	2	3.285	-	-	-	0.637	3.754	-	-	-	0.641
B-C	X70	2	3.977	-	-	-	0.618	4.534	-	-	-	0.624
			Cohesive soil deposit – Trench type TB					Cohesionless soil deposit – Trench type TB				
Soil	Steel Grade	h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.317	0.635	0.977	1.440	1.104	0.365	0.783	1.255	1.921	1.016
A-B	X65	1	0.349	0.734	1.161	1.757	1.088	0.452	1.086	1.870	3.053	0.994
A-B	X70	1	0.405	0.945	1.595	2.560	1.011	0.623	1.717	3.214	-	0.942
A-B	X60	2	0.345	0.738	1.182	1.808	1.090	0.395	0.713	1.531	2.444	1.009
A-B	X65	2	0.399	0.927	1.562	2.501	1.050	0.477	1.234	2.222	3.777	0.947
A-B	X70	2	0.518	1.436	2.697	4.766	0.931	0.594	1.565	2.849	4.892	0.958
A-C	X60	1	0.290	0.546	0.808	1.150	1.137	0.312	0.607	0.916	1.329	1.100
A-C	X65	1	0.316	0.617	0.934	1.357	1.118	0.342	0.692	1.071	1.589	1.086
A-C	X70	1	0.363	0.782	1.258	1.933	1.009	0.406	0.932	1.560	2.482	0.951
A-C	X60	2	0.310	0.615	0.938	1.374	1.141	0.334	0.699	1.103	1.665	1.080
A-C	X65	2	0.373	0.833	1.369	2.144	1.025	0.567	1.584	2.991	-	1.014
A-C	X70	2	0.454	1.148	2.039	3.426	0.972	0.526	1.489	2.832	-	0.884
B-C	X60	1	0.622	1.620	2.928	4.997	1.018	0.659	1.728	3.136	-	0.981
B-C	X65	1	0.695	1.922	3.605	-	0.978	0.808	2.377	4.634	-	0.943
B-C	X70	1	0.809	2.463	4.904	-	0.931	0.864	2.628	-	-	0.927
B-C	X60	2	0.667	1.846	3.467	-	0.998	0.802	2.443	4.866	-	0.930
B-C	X65	2	0.843	2.688	-	-	0.922	0.954	3.199	-	-	0.895
B-C	X70	2	1.078	3.989	-	-	0.854	1.313	-	-	-	0.788

5

1 **Table A15** Median peak ground velocity corresponding to the limit states, $PGV_{m,l}$, and total lognormal
 2 standard deviation, β_{tot} , for 1066.4 mm pipelines embedded in soil deposit of depth $H = 120$ m (- : the
 3 limit state is not reached).
 4

			Cohesive soil deposit – Trench type TA					Cohesionless soil deposit – Trench type TA				
Soil	Steel Grade	Burial depth, h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.575	1.675	3.244	-	0.823	0.667	1.970	3.849	-	0.787
A-B	X65	1	0.699	2.271	4.705	-	0.772	0.821	2.728	-	-	0.705
A-B	X70	1	0.877	3.296	-	-	0.700	1.013	3.848	-	-	0.660
A-B	X60	2	0.828	3.151	-	-	0.728	0.950	3.681	-	-	0.715
A-B	X65	2	1.113	-	-	-	0.659	1.275	-	-	-	0.653
A-B	X70	2	1.389	-	-	-	0.620	1.619	-	-	-	0.613
A-C	X60	1	0.488	1.312	2.420	4.207	0.842	0.534	1.425	2.616	4.526	0.830
A-C	X65	1	0.587	1.726	3.377	-	0.790	0.636	1.869	3.640	-	0.752
A-C	X70	1	0.697	2.308	4.840	-	0.734	0.772	2.567	-	-	0.687
A-C	X60	2	0.633	2.038	4.208	-	0.768	0.701	2.309	4.825	-	0.754
A-C	X65	2	0.834	3.174	-	-	0.696	0.923	3.579	-	-	0.666
A-C	X70	2	1.033	2.498	-	-	0.647	1.149	-	-	-	0.640
B-C	X60	1	1.322	-	-	-	0.752	1.349	-	-	-	0.743
B-C	X65	1	1.459	-	-	-	0.734	1.594	-	-	-	0.734
B-C	X70	1	1.744	-	-	-	0.701	2.050	-	-	-	0.681
B-C	X60	2	2.025	-	-	-	0.669	2.137	-	-	-	0.686
B-C	X65	2	2.832	-	-	-	0.622	2.850	-	-	-	0.618
B-C	X70	2	3.488	-	-	-	0.600	3.255	-	-	-	0.621
			Cohesive soil deposit – Trench type TB					Cohesionless soil deposit – Trench type TB				
Soil	Steel Grade	h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.243	0.457	0.674	0.958	1.189	0.282	0.535	0.796	1.138	1.164
A-B	X65	1	0.265	0.514	0.775	1.122	1.185	0.265	0.514	0.775	1.122	1.185
A-B	X70	1	0.299	0.631	1.002	1.521	1.106	0.342	0.722	1.147	1.742	1.073
A-B	X60	2	0.255	0.492	0.738	1.065	1.210	0.296	0.593	0.910	1.340	1.164
A-B	X65	2	0.290	0.599	0.937	1.405	1.188	0.290	0.599	0.939	1.407	1.189
A-B	X70	2	0.356	0.841	1.431	2.312	1.073	0.419	1.041	1.828	3.040	0.990
A-C	X60	1	0.221	0.396	0.567	0.785	1.230	0.245	0.437	0.625	0.864	1.217
A-C	X65	1	0.239	0.441	0.643	0.905	1.239	0.263	0.479	0.695	0.973	1.233
A-C	X70	1	0.267	0.526	0.800	1.169	1.168	0.284	0.542	0.809	1.161	1.190
A-C	X60	2	0.231	0.422	0.611	0.855	1.269	0.251	0.461	0.672	0.943	1.266
A-C	X65	2	0.272	0.548	0.845	1.251	1.208	0.301	0.622	0.974	1.460	1.173
A-C	X70	2	0.314	0.686	1.113	1.723	1.162	0.339	0.746	1.216	1.889	1.138
B-C	X60	1	0.482	1.262	2.288	3.915	0.976	0.578	1.619	3.060	-	0.890
B-C	X65	1	0.554	1.556	2.948	-	0.953	0.692	2.134	4.281	-	0.866
B-C	X70	1	0.646	2.005	4.041	-	0.898	0.873	3.164	-	-	0.777
B-C	X60	2	0.541	1.542	2.948	-	0.955	0.706	2.317	4.831	-	0.847
B-C	X65	2	0.681	2.225	4.629	-	0.888	0.913	3.505	-	-	0.775
B-C	X70	2	0.888	3.432	-	-	0.804	1.243	-	-	-	0.704

5

1 **Table A16** Median peak ground velocity corresponding to the limit states, $PGV_{m,l}$, and total lognormal
2 standard deviation, β_{tot} , for 1219.2 mm pipelines embedded in soil deposit of depth $H = 30$ m (- : the
3 limit state is not reached).
4

			Cohesive soil deposit – Trench type TA					Cohesionless soil deposit – Trench type TA				
Soil	Steel Grade	Burial depth, h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	1.288	4.523	-	-	0.774	1.449	-	-	-	0.775
A-B	X65	1	1.736	-	-	-	0.695	1.909	-	-	-	0.702
A-B	X70	1	2.353	-	-	-	0.667	2.217	-	-	-	0.669
A-B	X60	2	1.706	-	-	-	0.697	1.938	-	-	-	0.693
A-B	X65	2	2.100	-	-	-	0.684	2.192	-	-	-	0.697
A-B	X70	2	2.666	-	-	-	0.614	3.160	-	-	-	0.607
A-C	X60	1	1.349	-	-	-	0.771	1.401	-	-	-	0.768
A-C	X65	1	1.651	-	-	-	0.689	1.928	-	-	-	0.688
A-C	X70	1	2.176	-	-	-	0.656	2.247	-	-	-	0.657
A-C	X60	2	1.841	-	-	-	0.688	1.941	-	-	-	0.687
A-C	X65	2	2.143	-	-	-	0.691	2.270	-	-	-	0.685
A-C	X70	2	3.121	-	-	-	0.599	-	-	-	-	0.597
B-C	X60	1	-	-	-	-	-	-	-	-	-	-
B-C	X65	1	-	-	-	-	-	-	-	-	-	-
B-C	X70	1	-	-	-	-	-	-	-	-	-	-
B-C	X60	2	-	-	-	-	-	-	-	-	-	-
B-C	X65	2	-	-	-	-	-	-	-	-	-	-
B-C	X70	2	-	-	-	-	-	-	-	-	-	-
			Cohesive soil deposit – Trench type TB					Cohesionless soil deposit – Trench type TB				
Soil	Steel Grade	h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.526	1.251	2.140	3.373	1.050	0.580	1.446	2.544	4.109	1.008
A-B	X65	1	0.574	1.422	2.492	4.012	1.044	0.651	1.724	3.149	-	0.978
A-B	X70	1	0.645	1.722	3.159	-	0.996	0.744	2.144	4.126	-	0.925
A-B	X60	2	0.515	1.230	2.106	3.325	1.096	0.583	1.497	2.683	4.403	1.035
A-B	X65	2	0.589	1.517	2.723	4.472	1.066	0.727	2.153	4.153	-	0.990
A-B	X70	2	0.669	1.871	3.534	-	1.019	0.799	2.495	-	-	0.919
A-C	X60	1	0.527	1.319	2.327	3.767	1.011	0.553	1.412	2.522	4.125	0.987
A-C	X65	1	0.596	1.591	2.921	4.891	0.986	0.627	1.709	3.179	-	0.949
A-C	X70	1	0.687	2.010	3.907	-	0.932	0.728	2.187	4.318	-	0.892
A-C	X60	2	0.575	1.538	2.827	4.739	1.033	0.562	1.497	2.743	4.585	0.997
A-C	X65	2	0.621	1.742	3.298	-	1.003	0.665	1.953	3.802	-	0.953
A-C	X70	2	0.735	2.311	4.694	-	0.928	0.867	3.041	-	-	0.873
B-C	X60	1	1.508	-	-	-	0.793	2.062	-	-	-	0.738
B-C	X65	1	1.962	-	-	-	0.791	2.547	-	-	-	0.701
B-C	X70	1	2.207	-	-	-	0.766	3.076	-	-	-	0.666
B-C	X60	2	1.731	-	-	-	0.796	2.316	-	-	-	0.706
B-C	X65	2	2.120	-	-	-	0.766	2.847	-	-	-	0.665
B-C	X70	2	2.192	-	-	-	0.734	3.653	-	-	-	0.621

5

1 **Table A17** Median peak ground velocity corresponding to the limit states, $PGV_{m,l}$, and total lognormal
 2 standard deviation, β_{tot} , for 1219.2 mm pipelines embedded in soil deposit of depth $H = 60$ m (- : the
 3 limit state is not reached).
 4

			Cohesive soil deposit – Trench type TA					Cohesionless soil deposit – Trench type TA				
Soil	Steel Grade	Burial depth, h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	1.062	3.790	-	-	0.737	1.111	3.876	-	-	0.768
A-B	X65	1	1.352	-	-	-	0.681	1.369	-	-	-	0.724
A-B	X70	1	1.958	-	-	-	0.640	1.609	-	-	-	0.688
A-B	X60	2	1.342	-	-	-	0.683	1.387	-	-	-	0.722
A-B	X65	2	1.635	-	-	-	0.649	2.000	-	-	-	0.640
A-B	X70	2	2.090	-	-	-	0.604	2.486	-	-	-	0.602
A-C	X60	1	0.939	3.140	-	-	0.733	0.997	3.388	-	-	0.748
A-C	X65	1	1.401	4.041	-	-	0.696	1.201	4.524	-	-	0.694
A-C	X70	1	1.357	-	-	-	0.646	1.458	-	-	-	0.650
A-C	X60	2	1.111	1.267	-	-	0.697	1.250	-	-	-	0.697
A-C	X65	2	1.436	-	-	-	0.642	1.441	-	-	-	0.670
A-C	X70	2	1.754	-	-	-	0.610	1.857	-	-	-	0.619
B-C	X60	1	2.192	-	-	-	0.730	2.466	-	-	-	0.710
B-C	X65	1	2.562	-	-	-	0.671	3.619	-	-	-	0.677
B-C	X70	1	2.988	-	-	-	0.648	4.069	-	-	-	0.644
B-C	X60	2	2.784	-	-	-	0.664	3.435	-	-	-	0.662
B-C	X65	2	3.179	-	-	-	0.669	4.495	-	-	-	0.627
B-C	X70	2	4.364	-	-	-	0.611	-	-	-	-	-
			Cohesive soil deposit – Trench type TB					Cohesionless soil deposit – Trench type TB				
Soil	Steel Grade	h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.382	0.823	1.324	1.982	1.060	0.440	1.006	1.679	2.592	1.023
A-B	X65	1	0.417	0.932	1.532	2.338	1.074	0.489	1.174	2.019	3.197	1.013
A-B	X70	1	0.469	1.132	1.950	3.095	1.035	0.551	1.426	2.568	4.232	0.987
A-B	X60	2	0.372	0.793	1.267	1.885	1.150	0.449	1.072	1.838	2.904	1.010
A-B	X65	2	0.442	1.044	1.778	2.792	1.059	0.563	1.559	2.868	4.863	0.953
A-B	X70	2	0.487	1.223	2.160	3.501	1.098	0.598	1.698	3.237	-	1.006
A-C	X60	1	0.346	0.699	1.079	1.560	1.071	0.369	0.761	1.192	1.744	1.070
A-C	X65	1	0.368	0.755	1.178	1.718	1.114	0.405	0.872	1.402	2.098	1.068
A-C	X70	1	0.415	0.923	1.513	2.300	1.056	0.449	1.034	1.731	2.680	1.052
A-C	X60	2	0.344	0.688	1.057	1.522	1.206	0.366	0.772	1.227	1.816	1.096
A-C	X65	2	0.376	0.793	1.258	1.861	1.170	0.425	0.977	1.635	2.531	1.086
A-C	X70	2	0.412	0.920	1.511	2.304	1.171	0.472	1.159	2.021	3.240	1.120
B-C	X60	1	0.824	2.424	4.724	-	0.900	0.859	2.464	4.729	-	0.938
B-C	X65	1	0.937	2.926	-	-	0.881	0.909	2.655	-	-	0.937
B-C	X70	1	0.978	3.108	-	-	0.883	1.063	3.425	-	-	0.883
B-C	X60	2	0.818	2.423	4.744	-	0.934	0.772	2.109	3.926	-	1.011
B-C	X65	2	0.972	3.179	-	-	0.894	1.037	3.356	-	-	0.904
B-C	X70	2	1.110	3.943	-	-	0.852	1.009	3.228	-	-	0.937

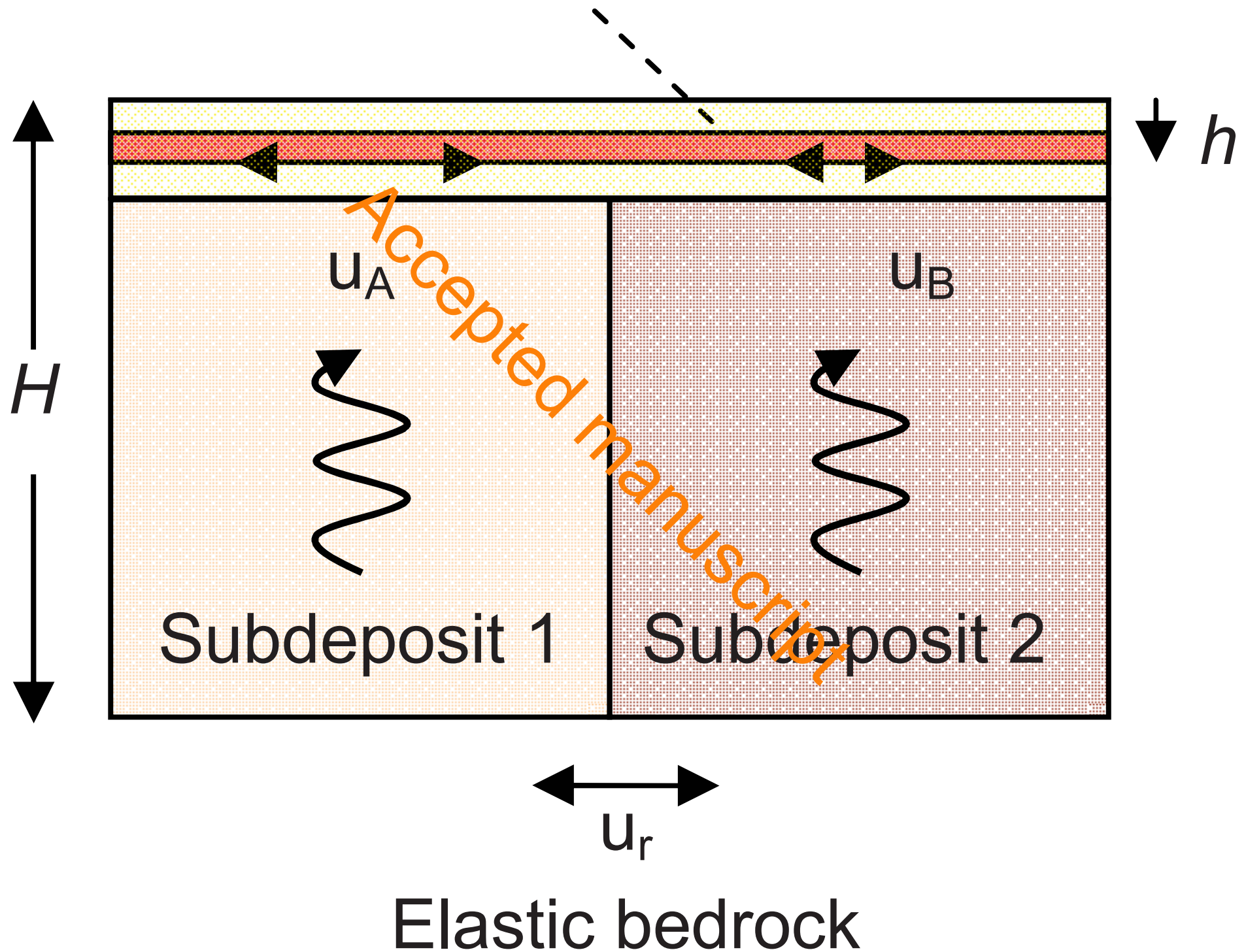
5

1 **Table A18** Median peak ground velocity corresponding to the limit states, $PGV_{m,l}$, and total lognormal
2 standard deviation, β_{tot} , for 1219.2 mm pipelines embedded in soil deposit of depth $H = 120$ m (- : the
3 limit state is not reached).
4

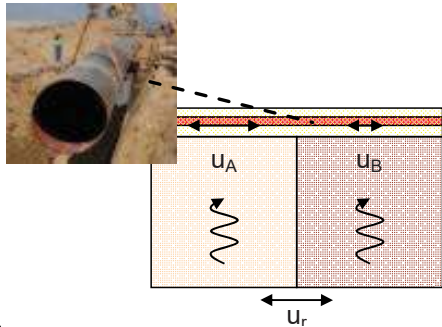
			Cohesive soil deposit – Trench type TA					Cohesionless soil deposit – Trench type TA				
Soil	Steel Grade	Burial depth, h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.783	2.573	-	-	0.764	0.882	2.877	-	-	0.734
A-B	X65	1	1.022	3.933	-	-	0.686	1.176	4.569	-	-	0.650
A-B	X70	1	1.464	-	-	-	0.650	1.392	-	-	-	0.620
A-B	X60	2	0.957	3.684	-	-	0.711	1.109	4.382	-	-	0.692
A-B	X65	2	1.304	-	-	-	0.641	1.421	-	-	-	0.671
A-B	X70	2	1.595	-	-	-	0.615	1.836	-	-	-	0.605
A-C	X60	1	0.664	2.030	4.050	-	0.759	0.729	2.219	4.418	-	0.732
A-C	X65	1	0.827	2.840	-	-	0.701	0.904	3.116	-	-	0.665
A-C	X70	1	0.938	3.823	-	-	0.652	1.062	4.050	-	-	0.628
A-C	X60	2	0.755	2.547	-	-	0.727	1.327	4.918	-	-	0.558
A-C	X65	2	0.988	3.023	-	-	0.670	1.128	4.667	-	-	0.629
A-C	X70	2	1.253	-	-	-	0.620	1.414	-	-	-	0.604
B-C	X60	1	1.694	-	-	-	0.736	1.764	-	-	-	0.720
B-C	X65	1	2.191	-	-	-	0.668	2.227	-	-	-	0.686
B-C	X70	1	2.571	-	-	-	0.643	2.969	-	-	-	0.633
B-C	X60	2	2.333	-	-	-	0.652	2.267	-	-	-	0.718
B-C	X65	2	2.672	-	-	-	0.662	3.071	-	-	-	0.624
B-C	X70	2	3.794	-	-	-	0.590	3.780	-	-	-	0.604
			Cohesive soil deposit – Trench type TB					Cohesionless soil deposit – Trench type TB				
Soil	Steel Grade	h (m)	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}	$PGV_{m,OLS}$ (m/s)	$PGV_{m,PILS}$ (m/s)	$PGV_{m,ULS}$ (m/s)	$PGV_{m,GLS}$ (m/s)	β_{tot}
A-B	X60	1	0.297	0.610	0.952	1.389	1.090	0.347	0.726	1.146	1.689	1.051
A-B	X65	1	0.322	0.680	1.079	1.597	1.089	0.417	0.851	1.323	1.924	1.076
A-B	X70	1	0.351	0.779	1.274	1.935	1.064	0.426	0.970	1.629	2.529	0.989
A-B	X60	2	0.291	0.588	0.910	1.318	1.153	0.340	0.715	1.133	1.675	1.091
A-B	X65	2	0.322	0.682	1.086	1.610	1.147	0.390	0.886	1.472	2.263	1.074
A-B	X70	2	0.359	0.814	1.352	2.079	1.121	0.434	1.057	1.831	2.920	1.013
A-C	X60	1	0.266	0.515	0.775	1.095	1.140	0.290	0.552	0.822	1.153	1.138
A-C	X65	1	0.282	0.551	0.833	1.183	1.164	0.306	0.587	0.879	1.239	1.158
A-C	X70	1	0.302	0.614	0.951	1.378	1.151	0.329	0.660	1.015	1.462	1.134
A-C	X60	2	0.262	0.502	0.750	1.055	1.238	0.279	0.527	0.780	1.089	1.235
A-C	X65	2	0.285	0.565	0.864	1.238	1.241	0.312	0.627	0.966	1.393	1.216
A-C	X70	2	0.314	0.661	1.048	1.548	1.203	0.339	0.714	1.131	1.672	1.208
B-C	X60	1	0.640	1.884	3.670	-	0.881	0.674	1.883	3.557	-	0.903
B-C	X65	1	0.751	2.408	4.950	-	0.832	0.788	2.402	4.787	-	0.869
B-C	X70	1	0.905	3.264	-	-	0.776	0.905	3.005	-	-	0.831
B-C	X60	2	0.669	2.065	4.147	-	0.867	0.756	2.371	4.809	-	0.876
B-C	X65	2	0.817	2.831	-	-	0.824	1.129	4.566	-	-	0.778
B-C	X70	2	1.017	4.057	-	-	0.740	1.111	4.405	-	-	0.807

5

Pipeline & surficial soil layer



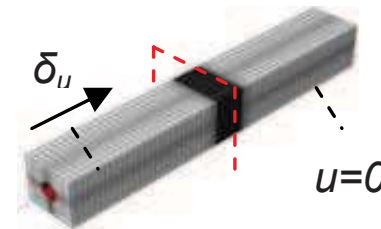
Selection of examined soil-pipe configurations



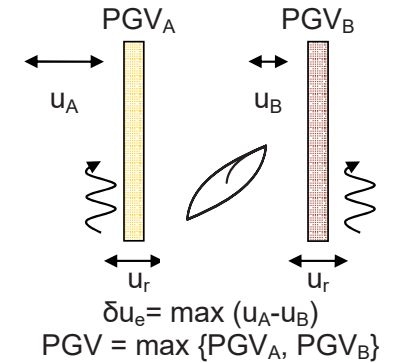
Selection of earthquake ground motions



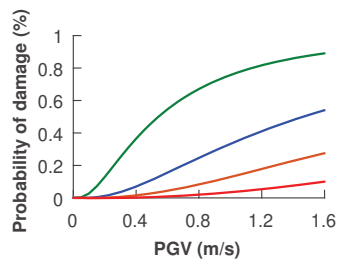
Computation of pipe response for an increasing ground displacement pattern, δ_u via a 3D SPI model



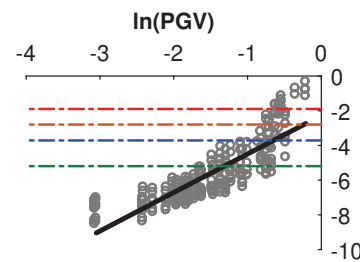
Computation of soil response via 1D soil response analyses



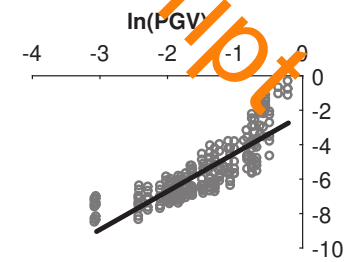
Development of PGV -based fragility curves for various soil-pipe configurations



Definition of limit states & uncertainties



Evolution of EDP with PGV at ground surface



Correlation of pipe response with soil response for various ground motions: δ_u - EDP

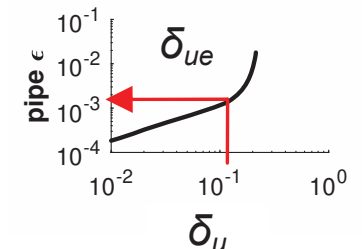
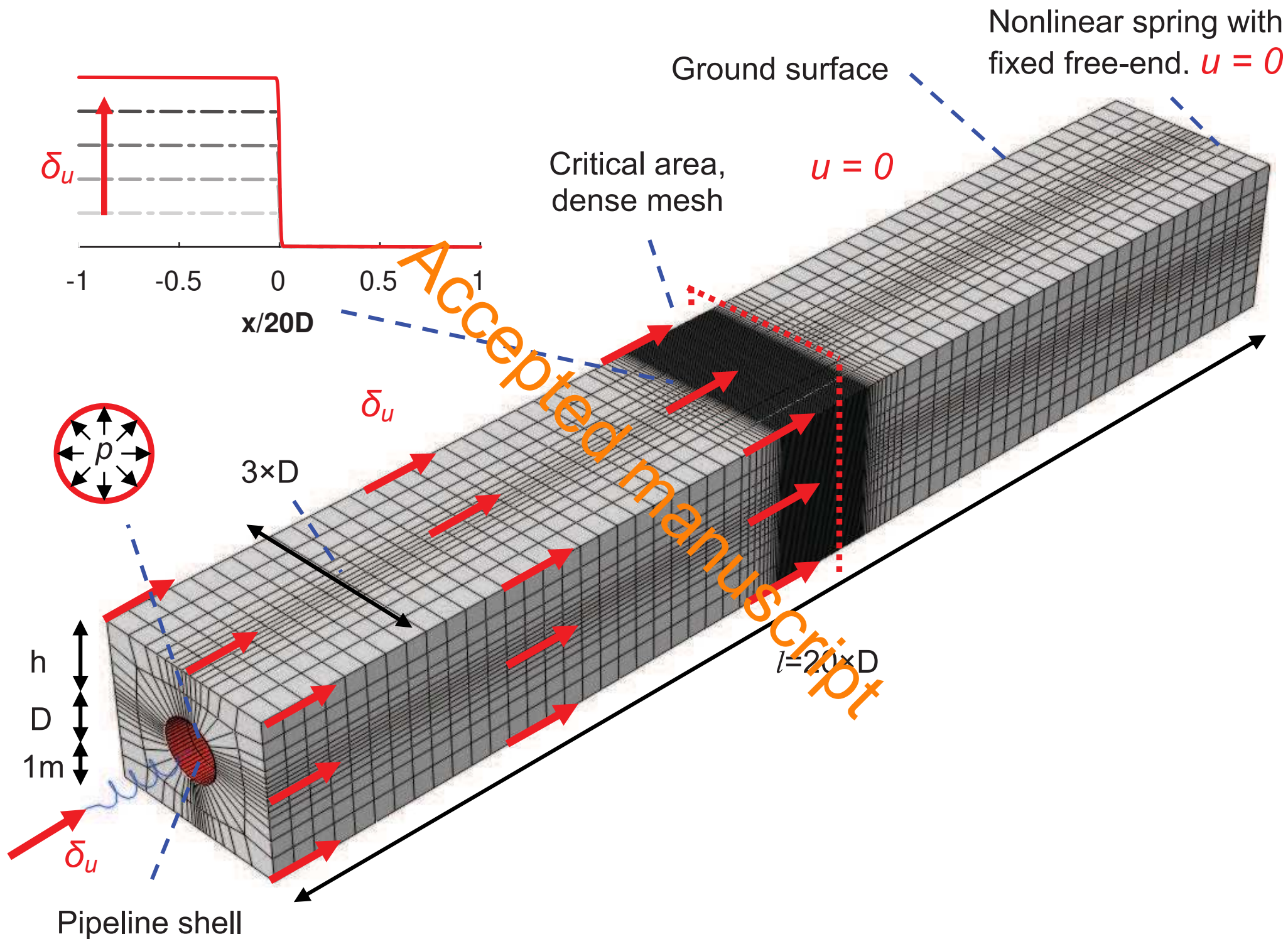
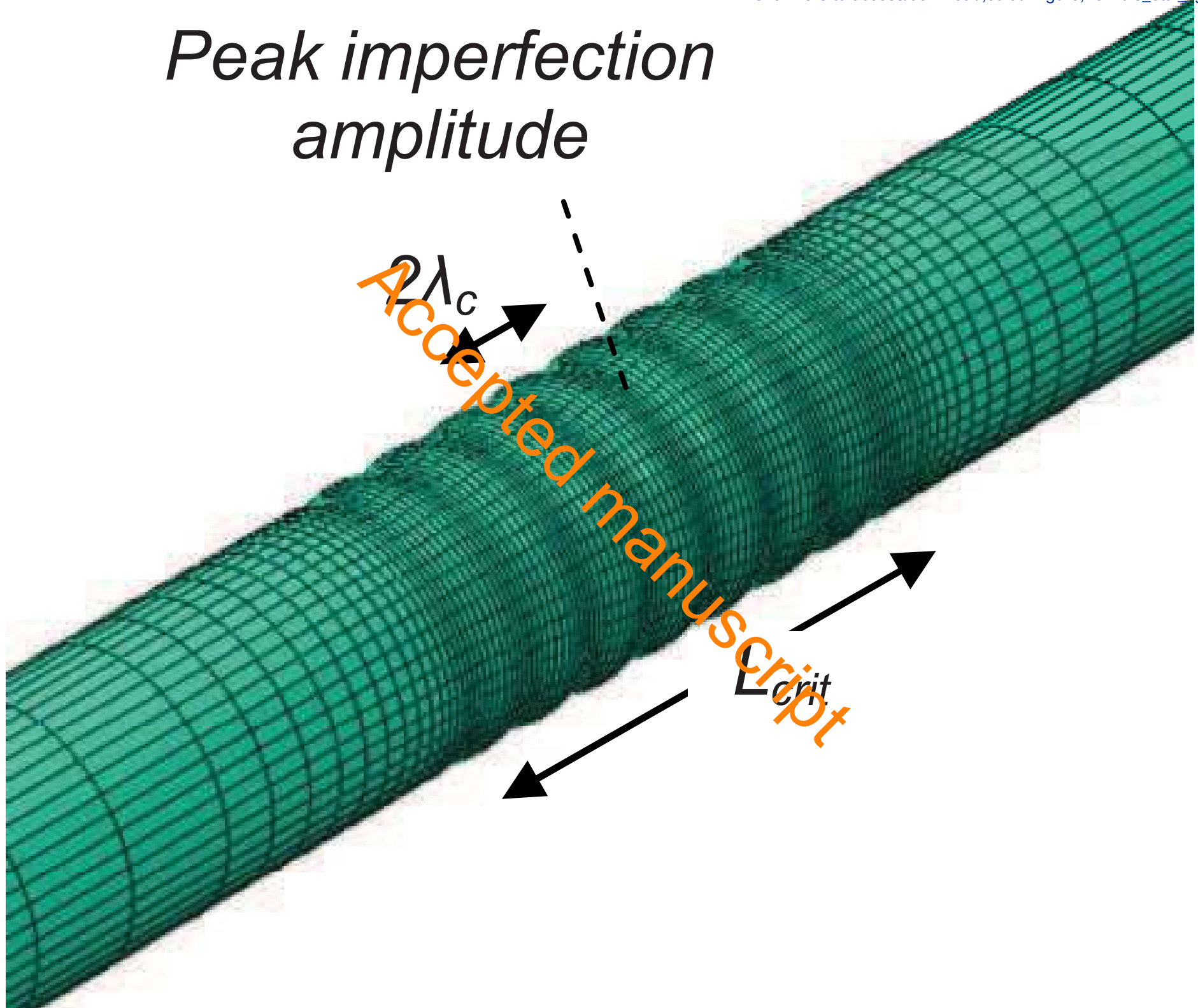


Figure 3



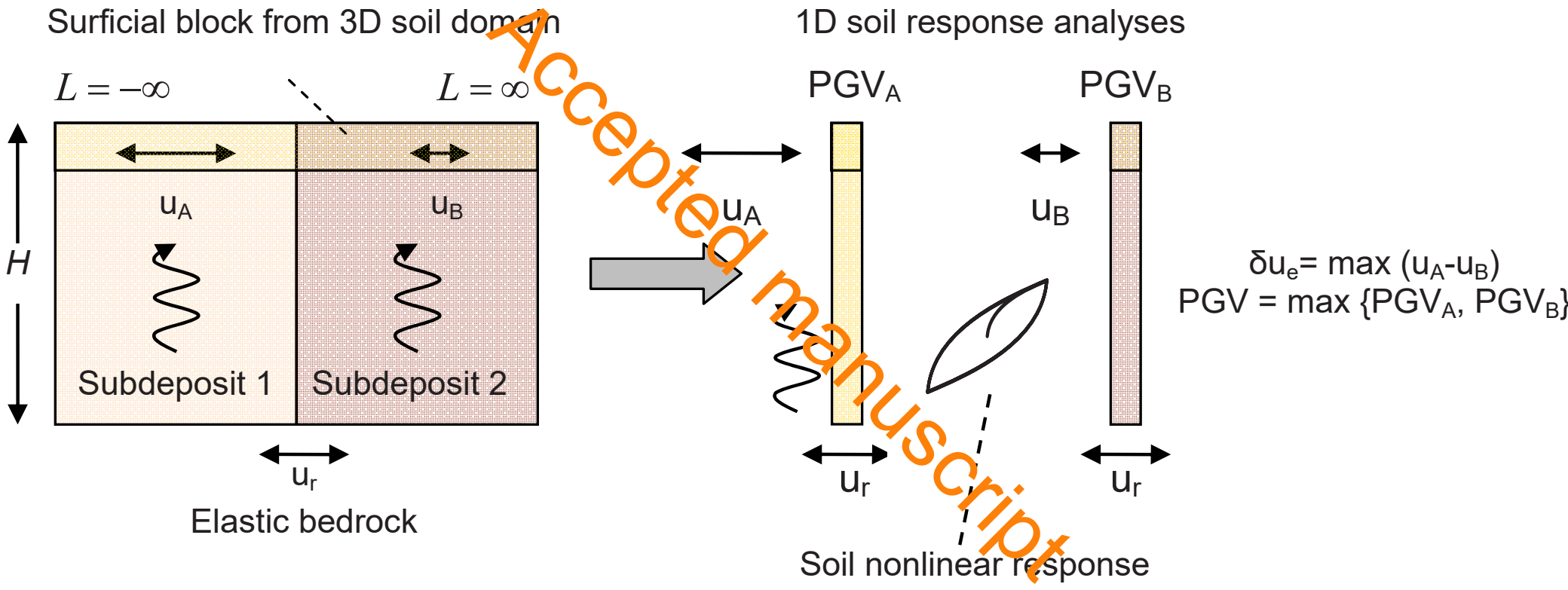
Peak imperfection amplitude

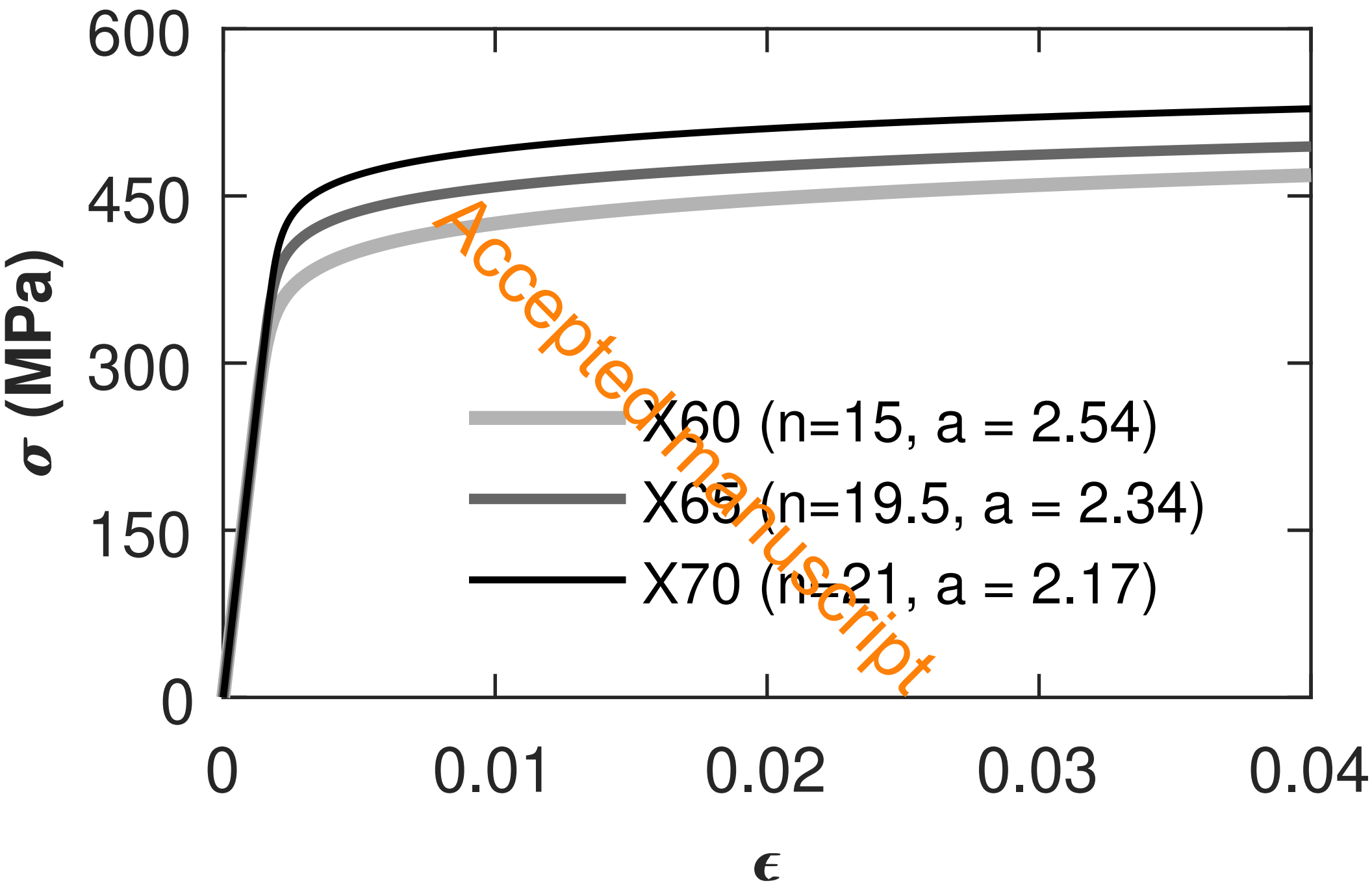


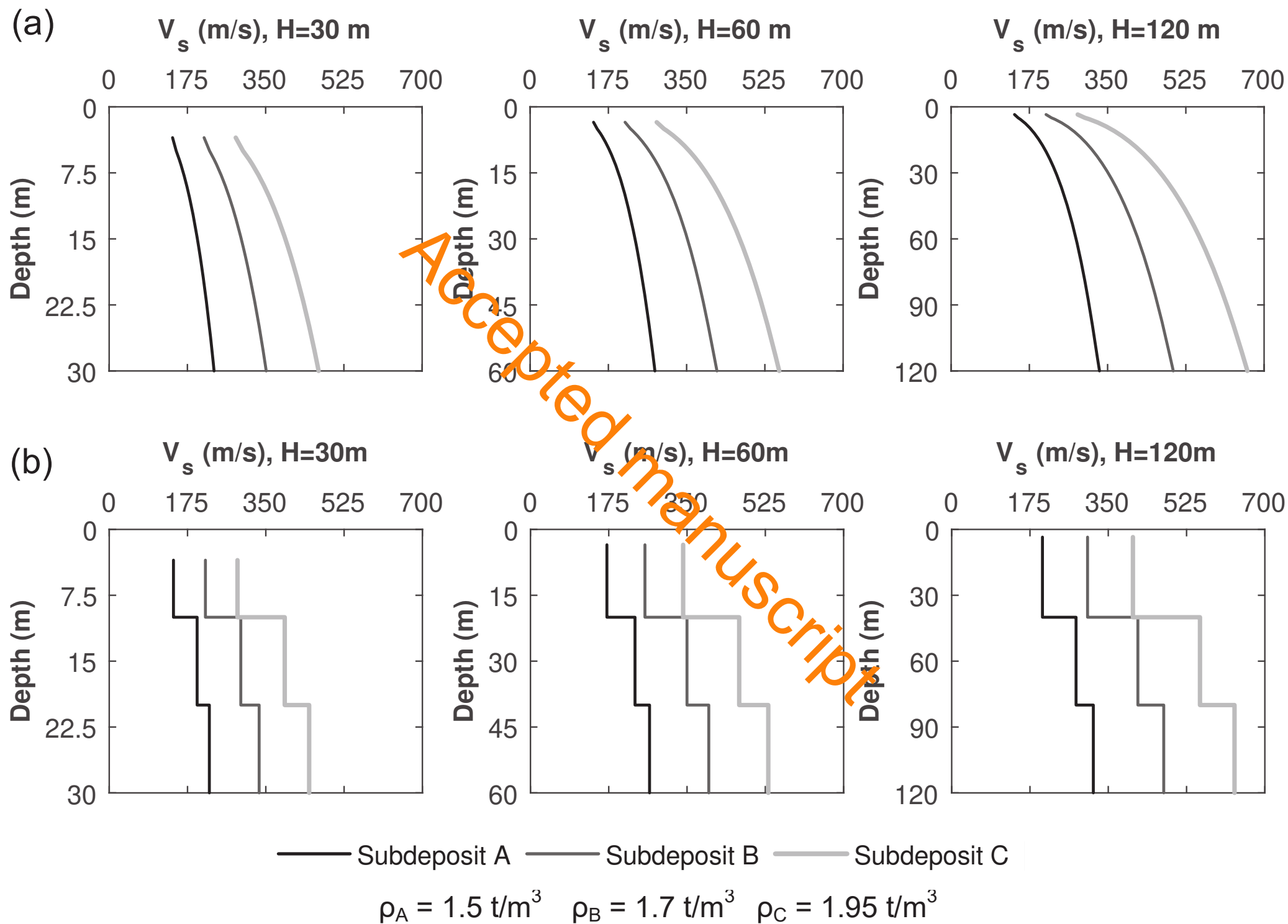
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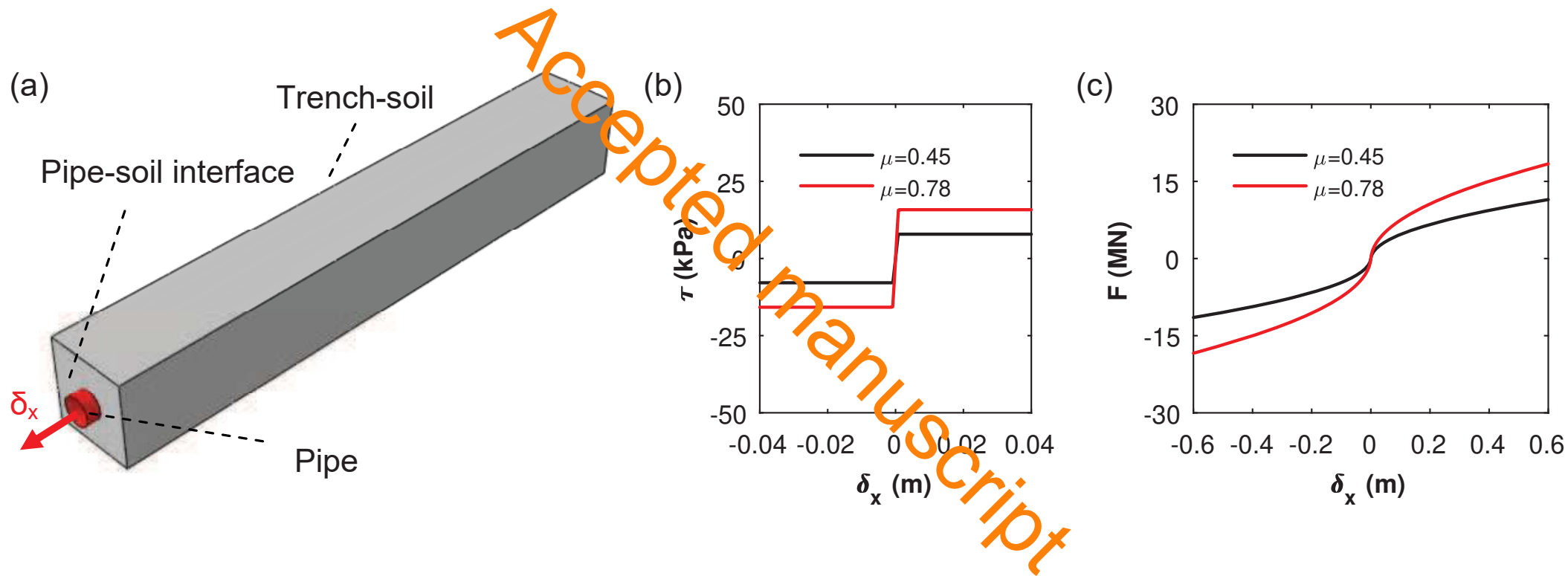
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Accepted manuscript



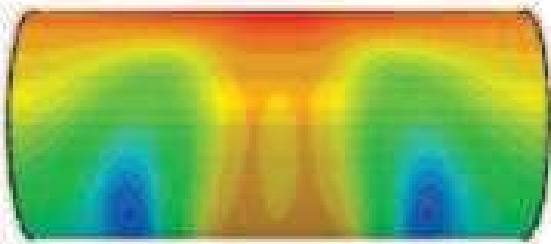




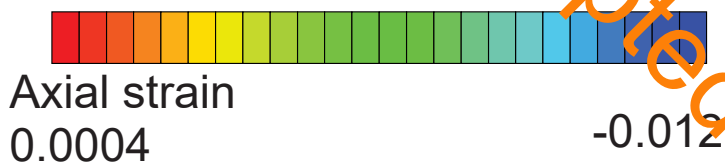
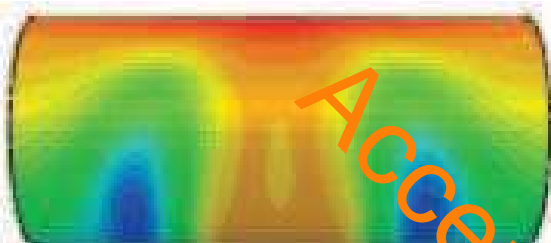


Trench TA ($\mu = 0.45$)

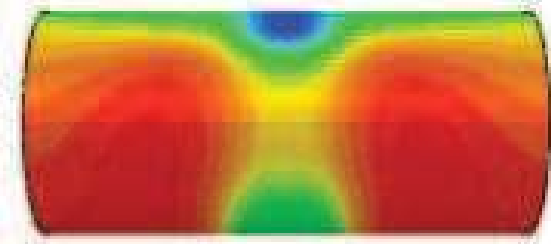
3D hybrid SPI model, L= 20D



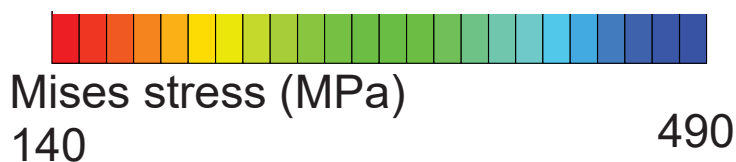
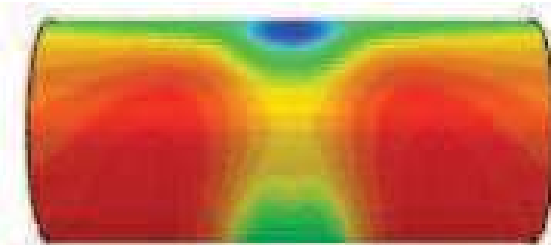
'Infinitely' long model, L=1000D



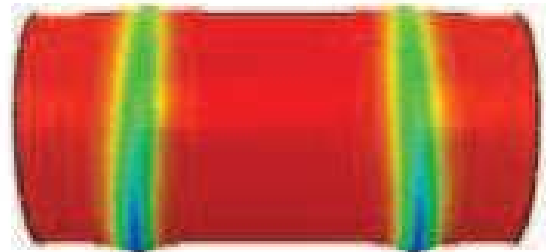
3D hybrid SPI model, L= 20D



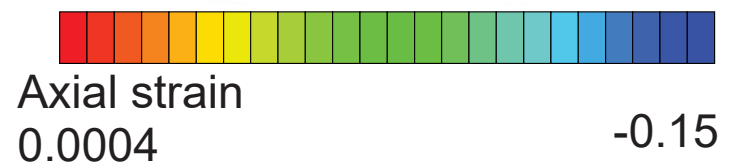
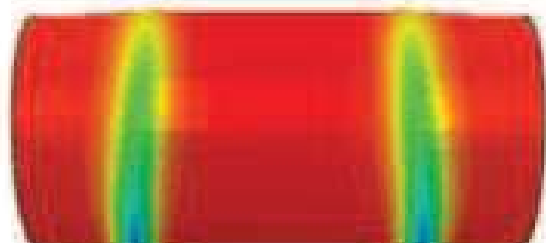
'Infinitely' long model, L=1000D

Trench TB ($\mu = 0.78$)

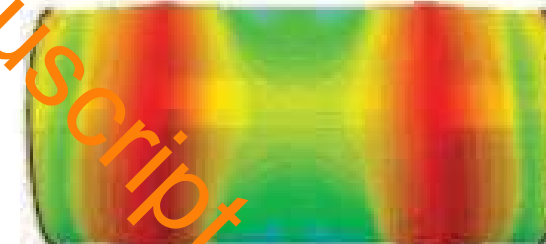
3D hybrid SPI model, L= 20D



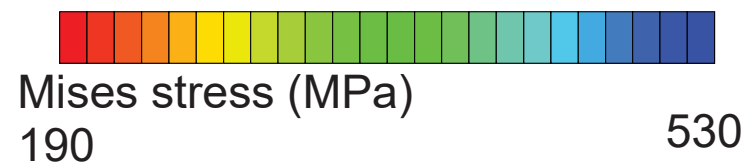
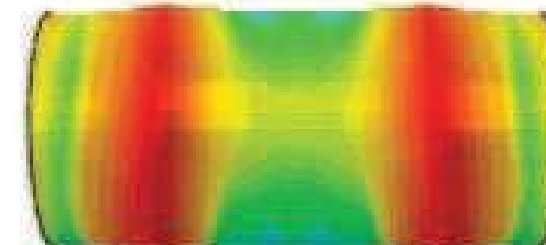
'Infinitely' long model, L=1000D



3D hybrid SPI model, L= 20D



'Infinitely' long model, L=1000D



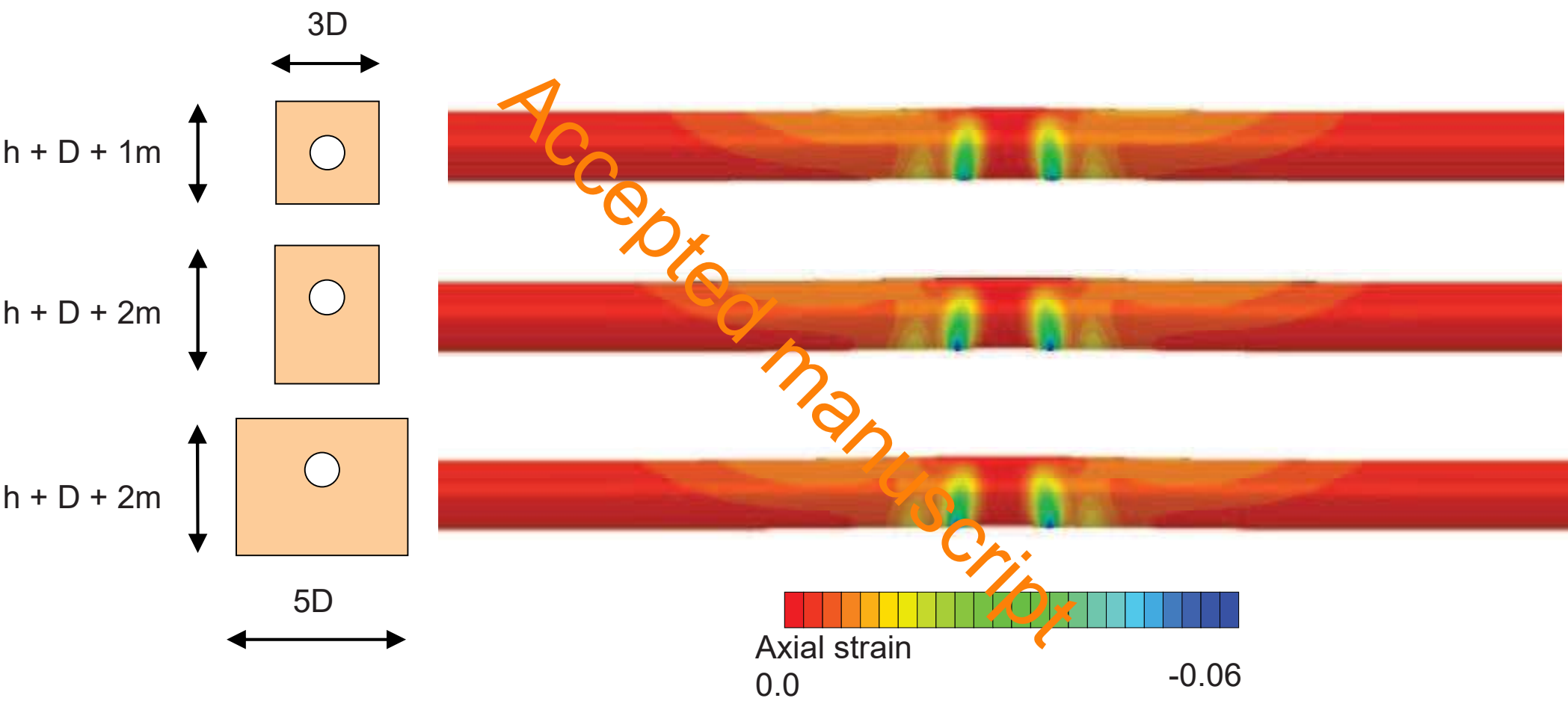
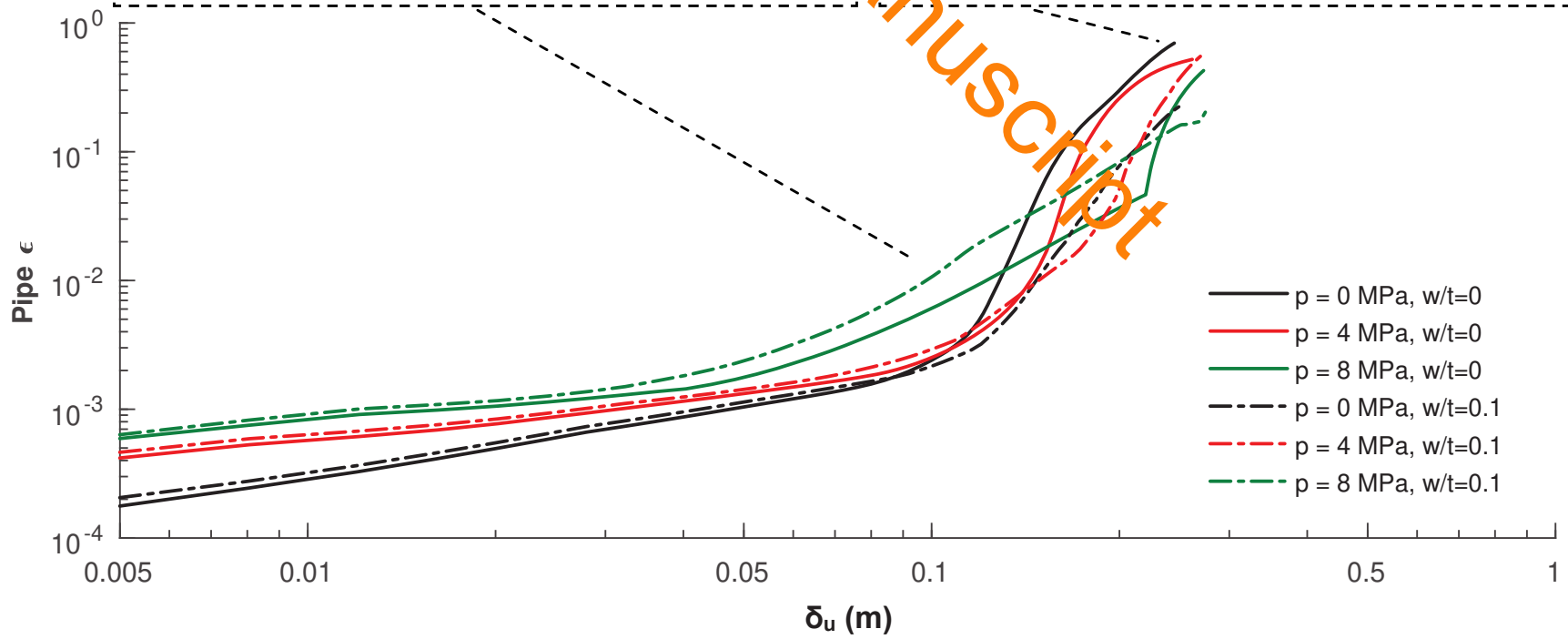
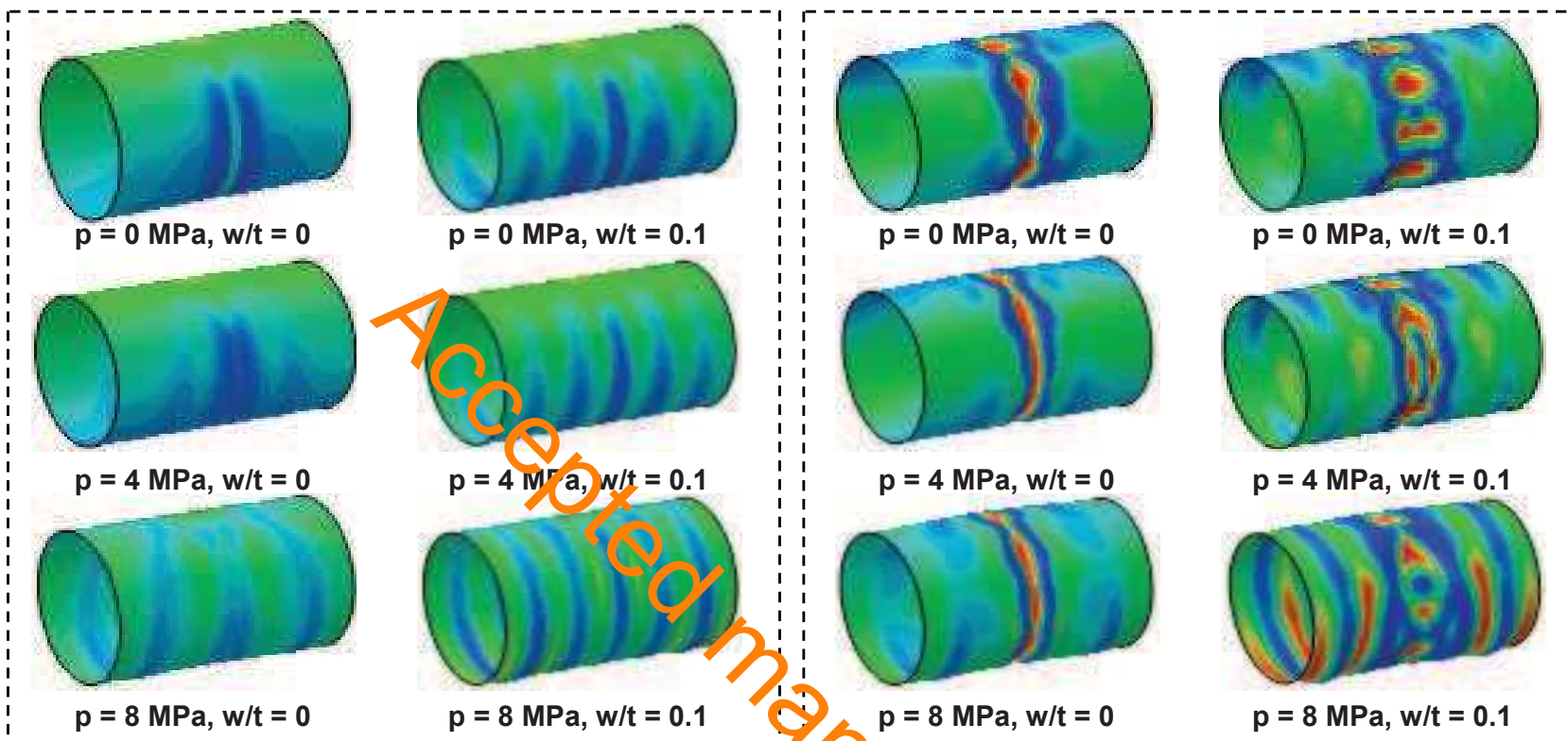
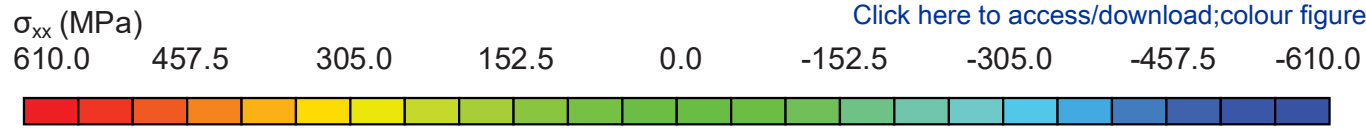
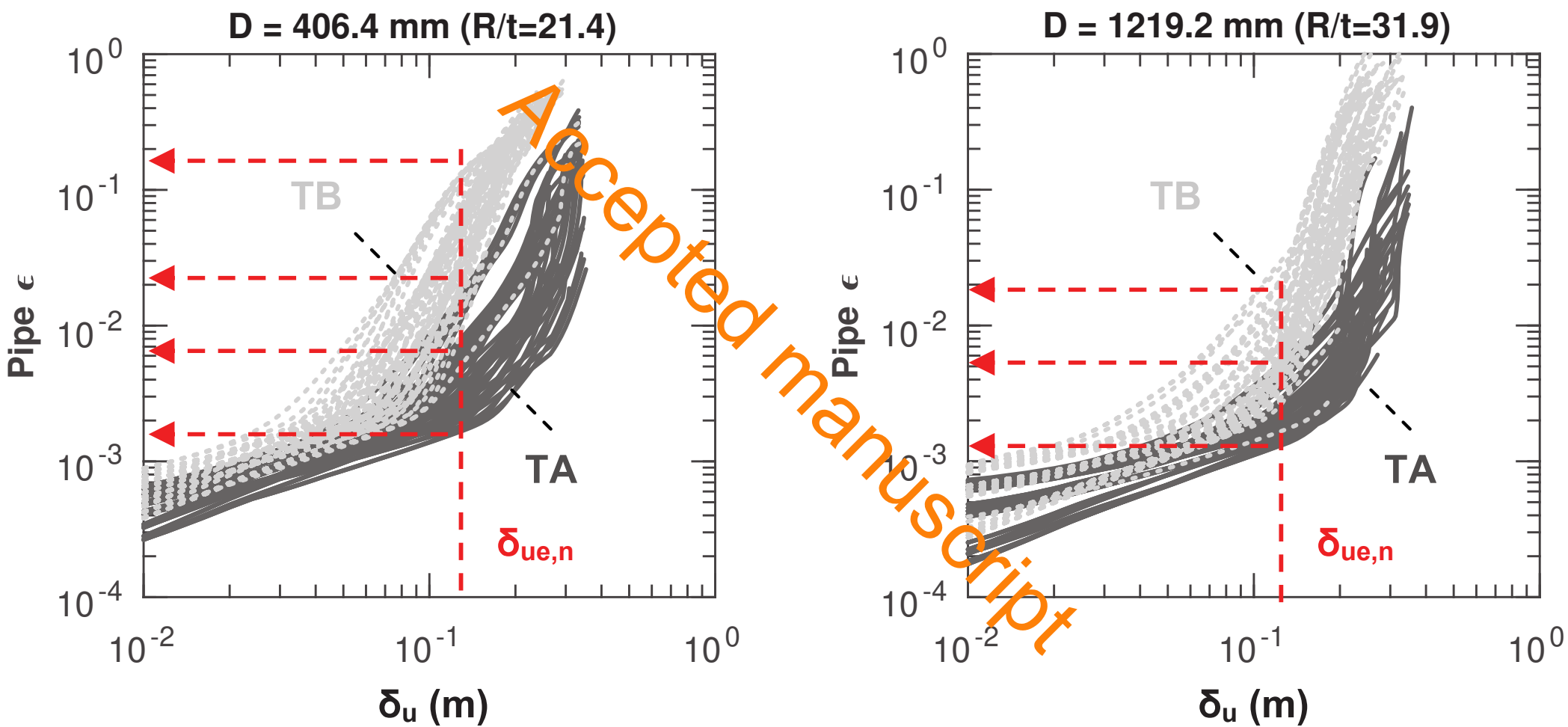
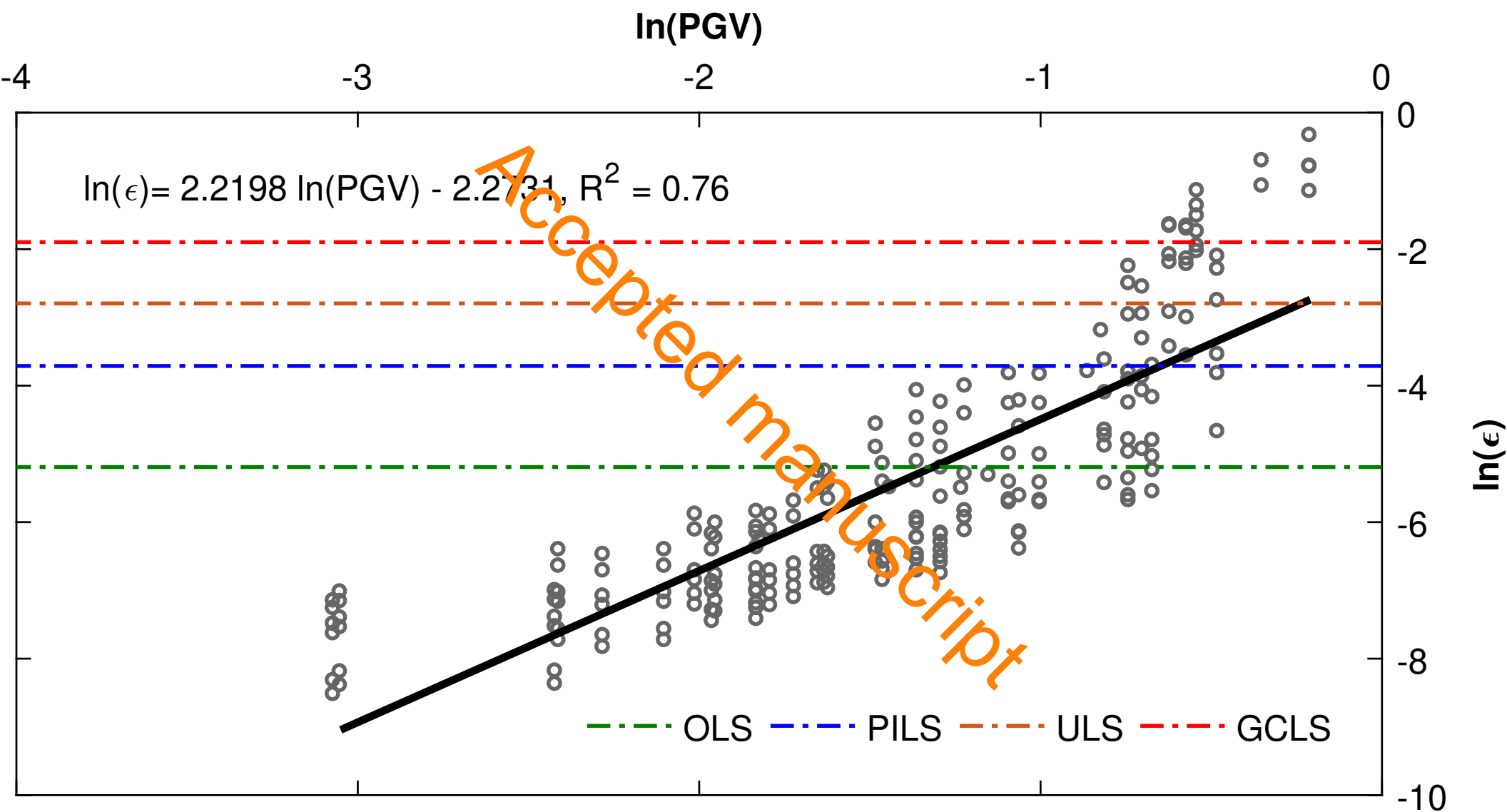


Figure 11







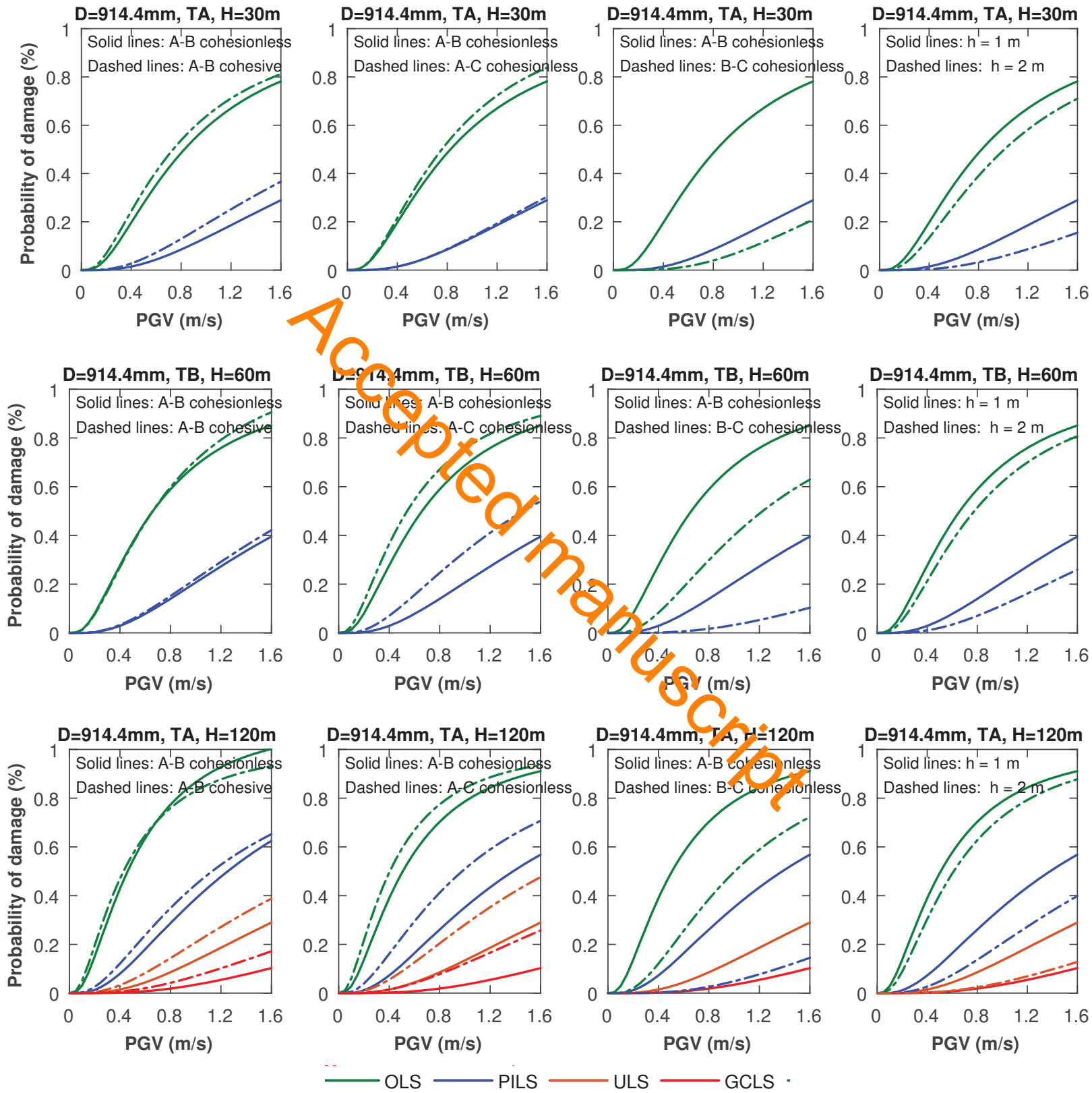
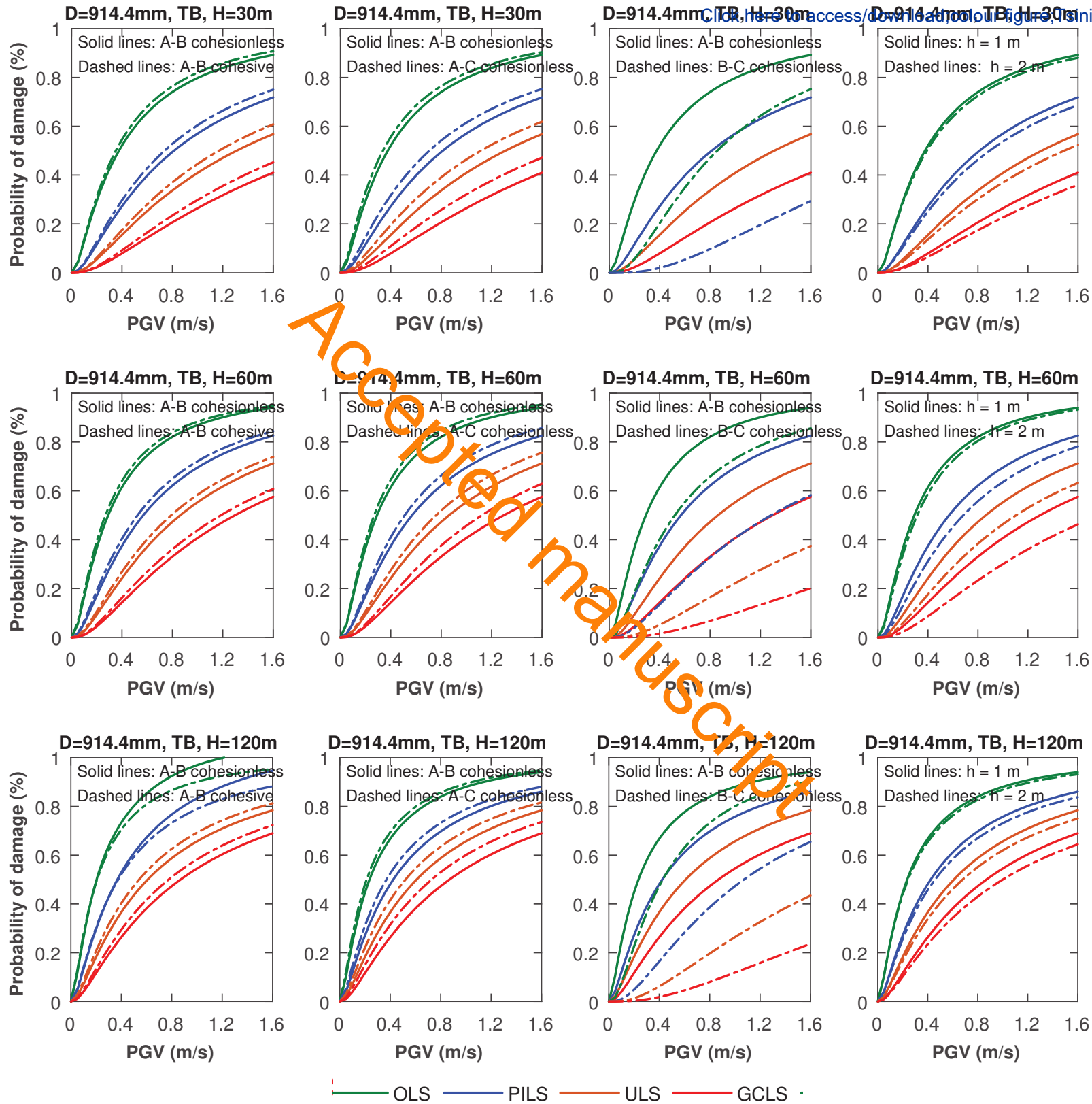
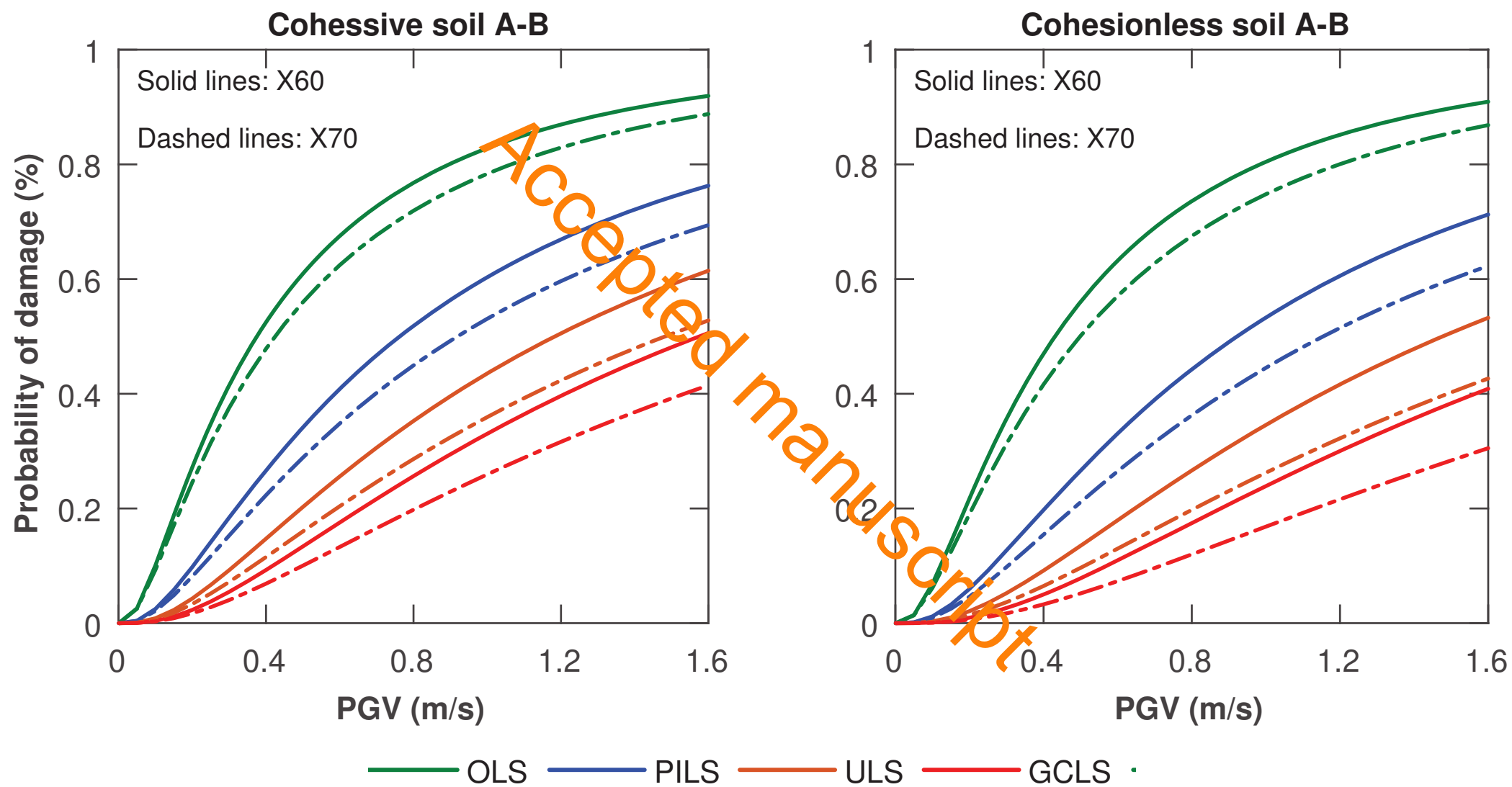
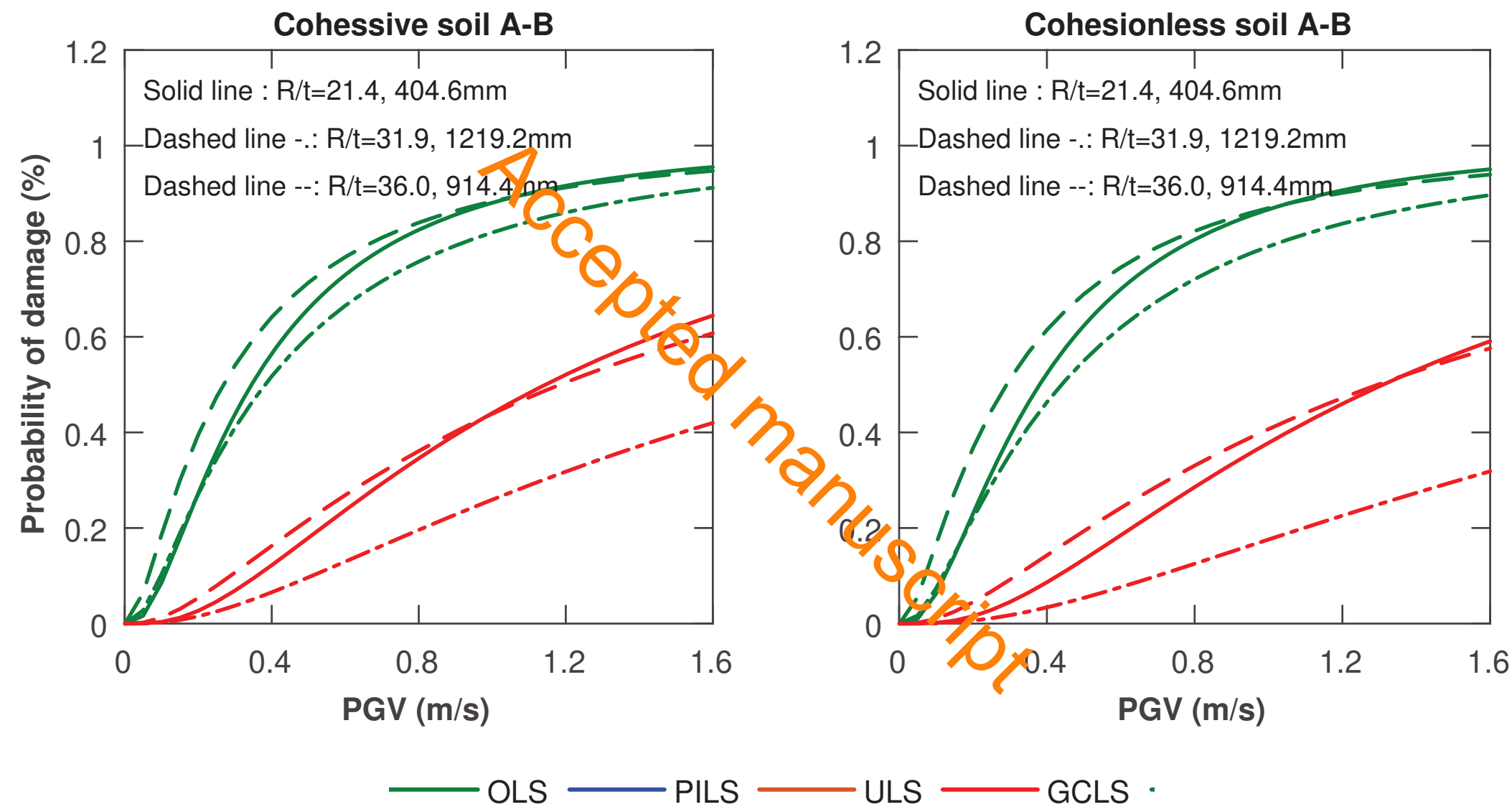


Figure 15







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