

1 ONSHORE BURIED STEEL FUEL PIPELINES AT FAULT 2 CROSSINGS: A REVIEW OF CRITICAL ANALYSIS AND 3 DESIGN ASPECTS

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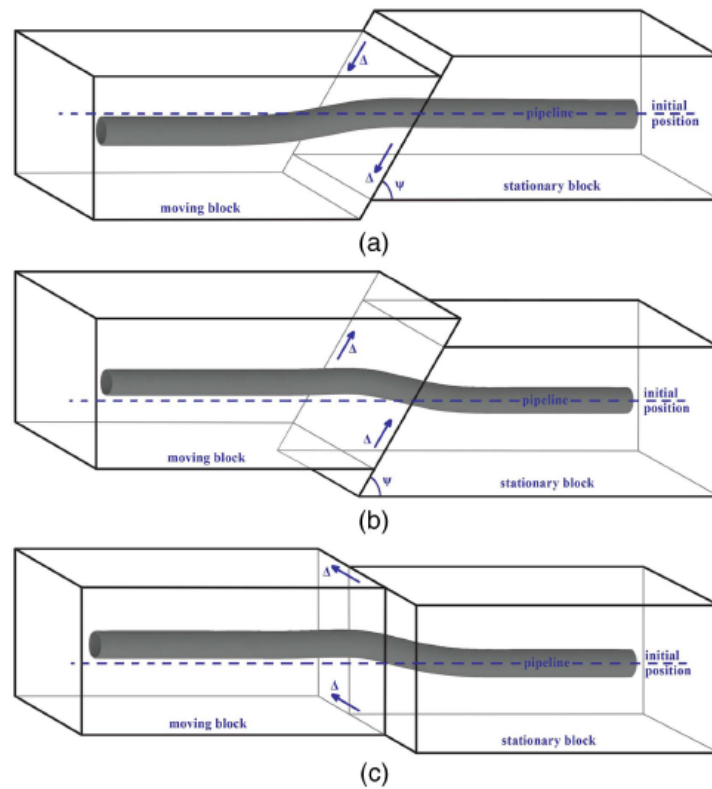
5 **Abstract:** Onshore buried steel pipeline infrastructure is a critical component of the fuel
6 supply system. Pipeline failure due to seismic actions is socially, environmentally, and
7 economically unacceptable and thus the design of pipelines at geohazard areas, such as fault
8 crossings, remains a hot topic for the pipeline community. There is an intense research effort
9 on the evaluation of the pipeline mechanical behavior and the strength verification at fault
10 crossings. Still, some aspects need in-depth consideration concerning practical applications. A
11 state-of-the-art review is presented on three critical analysis and design aspects, namely the
12 calculation of the design fault displacement via deterministic and probabilistic methods, the
13 effect of numerical modeling parameters such as soil spring properties, and the alternative pipe
14 protection measures in terms of availability, efficiency, and selection process. The critical
15 review offers a thorough insight on what is available and how to employ it in design, assisting
16 engineers and pipe operators in improving pipe safety.

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21 **Author Keywords:** buried pipeline; fault crossing; design fault displacement; numerical
22 modeling; protection measures

23 **Introduction**

24 Onshore buried continuous steel pipelines are the essential link in the fuel supply system,
25 interconnecting wells, storage facilities, process plants, and customers. Pipes have been proven
26 to be the cheapest and most efficient mode of transporting oil and gas for almost 100 years
27 (Strogen et al. 2016). Securing the integrity of pipeline networks against seismic actions with
28 state-of-the-art tools (Fragiadakis et al. 2015) is crucial for the safety and prosperity of
29 communities. A pipe failure could be disastrous for society, the environment, and the economy
30 due to injuries or fatalities, air/soil/water contamination, monetary losses, and downtimes,
31 respectively (Nair et al. 2018; Papadakis 1999). Seismic fault activation is the most catastrophic
32 seismic-induced action on pipelines (Girgin and Krausmann 2016). The fault mechanism,
33 namely normal, reverse, or strike-slip, is the dominant parameter affecting the pipe response
34 because it determines the pipe deformation (Fig. 1). Pipe tension and pipe compression are
35 predominant in cases of normal and reverse faulting, respectively while bending dominates the
36 pipe behavior in case of strike-slip faulting (O'Rourke and Liu 2012). Modern codes (Table 1)
37 provide recommendations and guidelines for the design and assessment of pipes at fault
38 crossings.



39

40 Fig. 1. Pipeline deformation caused by (a) normal; (b) reverse; and (c) strike-slip fault rupture
 41 (the left block is the moving and the right is the stationary one).

42 The potential failure modes of a pipeline subjected to faulting are local buckling of the
 43 pipeline wall, tensile rupture, cross-section ovalization, and beam-mode buckling in the case
 44 of reverse faulting. Strength verification of pipes under faulting (displacement-controlled
 45 loading) against local buckling and tensile fracture is carried out in strain terms. Mohr (2003)
 46 justifies the adoption of strain-based performance limits through an example: if two pipes of
 47 different steel grades are fitted to a curved ground surface, then the same strain level would be
 48 developed, unlike the stress level because strains are unambiguously defined by the ground
 49 curvature.

50 The compressive strain shall be limited to ensure that local buckling of the pipeline wall
 51 is avoided. The concentration of compression leads to the initiation of a wrinkle that neither
 52 interrupts fuel flow nor allows a leak. Further increase of compression allows the evolvement

53 of the wrinkle to a local buckle, stating a limit state exceedance (Houliara and Karamanos 2006;
54 Karamanos 2002; Kyriakides and Corona 2007). Parameters affecting the compressive strain
55 capacity of a pipe are (1) the girth welds that might “attract” the buckle to a nearby region
56 (Dorey et al. 2000; Prion and Birkemoe 1992), (2) the pressurization that relieves the
57 compression as it creates tensile hoop stresses preventing cross-section distortion (Greiner and
58 Guggenberger 1998; Limam et al. 2010), and (3) the external pressure that reduces the
59 resistance to local buckling (Vasilikis and Karamanos 2011) and might contribute to buckling
60 propagation (Corona and Kyriakides 1991, 2000). The seminal work of Gresnigt (1986) has
61 contributed to the development of limiting expressions for the compressive strain.

62 Tensile strains may cause rupture of the pipeline wall at areas of strain concentration.
63 Girth welds between two adjacent pipeline segments are considered to be the weak link due to
64 the imperfections associated with the welding process (Wang et al. 2004), the reduced ductility
65 in the heat-affected zone, the potential tolerable defects, and the potential corrosion due to
66 coating on-site (Abdulhameed et al. 2016; O’Rourke and Liu 2012). Note that apart from the
67 recent PRCI Guidelines that are based on the work of Liu et al. (2012a; b) and Wang et al.
68 (2012), typically high-quality welding and a defect-free homogenous pipe material (Liu et al.
69 2009) is assumed in the code expressions for the tensile strain capacity.

70 The crossing pipeline is subjected to significant compression in the case of reverse
71 faulting, leading to potential pipe local or global buckling. The latter is denoted also as
72 upheaval buckling or beam-mode buckling and is typically defined as the exposure of the pipe
73 on the ground surface (Demirci et al. 2018; Rofooei et al. 2018; Rojhani et al. 2012). Pipe
74 global buckling caused by fault movement has been reported by Koch (1933) at the Buena
75 Vista Hills Oil Field in California, USA, and by Hamada and O’Rourke (1992) after the 1964
76 Niigata, Japan earthquake. Global buckling might not result in a failure but local buckling
77 might follow shortly after (Liu et al. 2017; Xu and Lin 2017). Whether the pipe will buckle

78 locally on globally depends on the local slenderness (diameter to thickness ratio D/t) and the
79 burial depth. Deeply buried pipes with a high D/t ratio tend to buckle locally, while shallowly
80 buried ones with low D/t ratio tend to buckle globally (Melissianos et al. 2020; Yun and
81 Kyriakides 1990). Specific guidelines for the assessment and protection of pipes against global
82 buckling are not provided in design codes, apart from the general requirement of CSA Z662
83 (Canadian Standards Association 2019), for pipe proper design in case the global buckling is
84 harmful to the pipe.

85 The improvement of pipeline safety against seismic hazards remains a hot topic for the
86 pipeline community and thus there is a significant ongoing research effort worldwide. The
87 structural health assessment of pipelines under faulting is carried out using analytical tools (e.g.
88 Karamitros et al. 2011; Sarvanis and Karamanos 2017; Talebi and Kiyono 2020), numerical
89 modeling (e.g. Banushi et al. 2018; Demirci et al. 2018; Trifonov 2015; Vazouras et al. 2015),
90 and experimental testing (e.g. Fadaee et al. 2020; Ha et al. 2010; Moradi et al. 2013; O'Rourke
91 et al. 2016; Sarvanis et al. 2018; Tsatsis et al. 2019; Xie et al. 2013). Still, three analysis and
92 design aspects require more attention by designers, operators, and researchers: (1) the
93 calculation of the design fault displacement either via simplified deterministic approaches or
94 full probabilistic fault displacement hazard analysis, (2) the critical details of the numerical
95 modeling techniques that affect the reliability of the calculations, and (3) the alternative seismic
96 countermeasures for the pipe protection. The critical state-of-the-art review of this study
97 presents the available knowledge on these topics and mainly highlights the essential parameters
98 that engineers and pipe operators should bear in mind to improve pipe safety and reliability. At
99 the same time, the review shows the fields that are open to further research.

100 **Design Fault Displacement**

101 Structural codes provide a straightforward procedure for the estimation of the seismic loads for
102 buildings and other above-ground structures, for example, EN 1998-1 (European Committee
103 for Standardization 2004) and ASCE/SEI 7-10 (American Society of Civil Engineers 2010).
104 On the other hand, pipeline codes lack specific provisions for the calculation of the design fault
105 displacement, which is typically calculated via empirical fault scaling relations, featuring a
106 deterministic approach. Davis (2008) has adopted the Wells and Coppersmith (1994) relations
107 to estimate the design fault displacement (Δ) for water pipelines crossing tectonic faults. The
108 displacement is estimated via the characteristic or the maximum earthquake magnitude (M)
109 that is obtained from disaggregation results of a Probabilistic Seismic Hazard Analysis. Then,
110 the displacement value is adjusted via correction factors (c) to account for the fault activity
111 and the pipe function class. Briefly, the design fault displacement is estimated as (Thompson
112 et al. 2018):

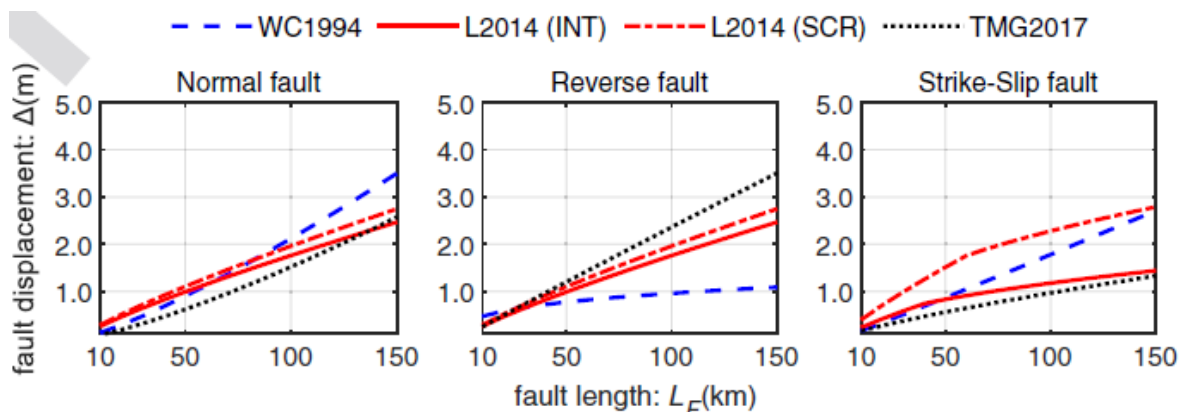
$$\Delta = c\Delta^*(M) \quad (1)$$

113 where Δ^* is either the maximum or the average fault displacement computed via the relation:

$$\Delta^* = a + bM \quad (2)$$

114 The use of empirical fault scaling relations is discussed indicatively by Dijkstra et al.
115 (2021), O'Rourke and Liu (2012), and ASCE Guidelines (American Society of Civil Engineers
116 2011). Most engineers are familiar with the 30 years old relations of Wells and Coppersmith
117 (1994), while recently Leonard (2014), and Thingbaijam et al. (2017) have published new
118 relations using advanced statistical methods. These sets of expressions include relationships
119 among fault characteristics and associated earthquakes, such as fault displacement, fault length,
120 fault width, earthquake magnitude, etc. Most of these characteristics are specialized
121 seismological information that engineers are not familiar with, apart from the fault length (L_F)

122 that can be obtained from a tectonic map or an available seismic hazard model. In such case,
 123 the fault displacement could be estimated via the (fault scaling) relation $\Delta \sim f(L_F)$ with respect
 124 to the fault mechanism, as presented in Table 2, where Δ is the average surface displacement
 125 and L_F is the fault length reported in geological or seismic hazard maps, being typically the
 126 subsurface. It should be noted that the Wells and Coppersmith (1994) estimation of the average
 127 displacement is based on the surface length (SL) that is typically lower than the subsurface
 128 length. In this case, the transformation $SL = 0.75L_F$ is used, as Wells and Coppersmith (1994)
 129 found that, on average, the surface rupture equals 75% of the subsurface rupture. Moreover,
 130 Leonard (2014) and Thingbaijam et al. (2017) provide a relation between the subsurface
 131 average fault displacement (Δ_{sub}) and the subsurface fault length. Based on the mode of the
 132 distribution ratios of average subsurface displacement to average surface displacement
 133 calculated by Wells and Coppersmith (1994), the transformation $\Delta_{sub} = 1.32\Delta$ is used to
 134 estimate the average surface fault displacement. A comparison of alternative empirical
 135 relations $\Delta \sim f(L_F)$ is presented in Fig. 2 within a fault length range of $10\text{km} \leq L_F \leq 150\text{km}$.
 136 As expected, a non-negligible variation is observed in the estimation of Δ values because each
 137 set of relations has been created using a different database, employing a different statistical
 138 method, and relying or not on a physical model (Wang 2018).



139
 140 Fig. 2. Average fault displacement versus fault length via fault scaling relations. [Note:
 141 WC1994: Wells and Coppersmith (1994), L2014: Leonard (2014), TMG2017: Thingbaijam et

142 al. (2017), INT: interplate tectonic environment, and SCR: stable continental region tectonic
143 environment.]

144 The estimation of the design fault displacement via fault scaling relations (deterministic
145 approach) leads to an unknown level of conservatism and safety (Bommer 2002) because
146 primarily the recurrence of the fault displacement is neglected. This recurrence might be taken
147 into account indirectly via factors in a deterministic approach but still, it is a rough approach
148 with many unquantified uncertainties. Additionally, there is no clear evidence, guidelines, or
149 specific recommendations on using one over the other set of empirical relations for pipe design.
150 In any case, these relations could provide a useful preliminary estimation of the displacement
151 that the fault at hand might undergo based on its dimensions.

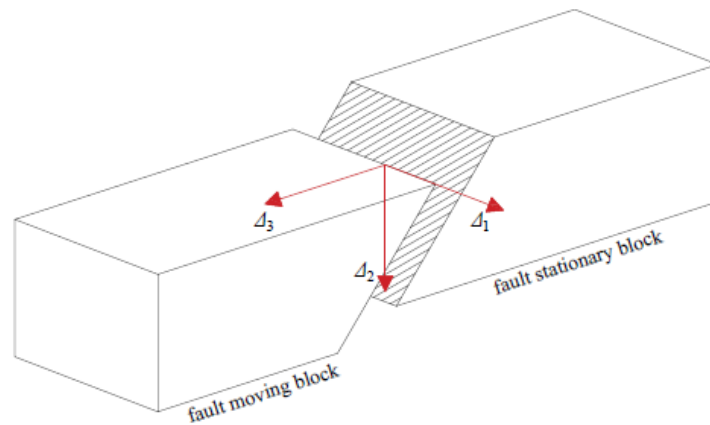
152 Pipeline networks are critical infrastructure and a performance-based framework
153 (Cornell and Krawinkler 2000) is required to satisfy the resilience requirements (United
154 Nations 2015). The full probabilistic treatment of the pipe–fault crossing problem in a rigorous
155 scheme is a complex task and some attempts to work around the problem have been carried
156 out. Strom et al. (2011) estimated the annual rate of rupturing displacement for pipeline design
157 as the product of earthquake occurrence, surface rupturing during the earthquake, and pipeline
158 being intercepted by the rupture probabilities. Cheng and Akkar (2017) have discussed the
159 probabilistic fault displacement hazard via Monte Carlo Simulation. Recently, Ni et al. (2020)
160 employed the Lasso regression, a machine learning technique, to develop fragility curves for
161 pipes at fault crossings. A comprehensive framework for the performance-based assessment of
162 pipelines at fault crossings has been presented by Melissianos et al. (2017b, 2021) using the
163 Probabilistic Fault Displacement Hazard Analysis (PFDHA) of Youngs et al. (2003). The latter
164 is an appropriate tool to quantify the probabilistic nature of earthquake faulting. The analysis
165 provides the fault displacement hazard on the pipeline crossing site, namely the mean annual
166 frequency of exceeding (λ_{Δ}) predefined fault displacement values (δ) via the expression:

$$\lambda_{\Delta}(\delta) = \nu_F \sum_i P(\Delta > \delta | m_i) P_M(m_i) \quad (3)$$

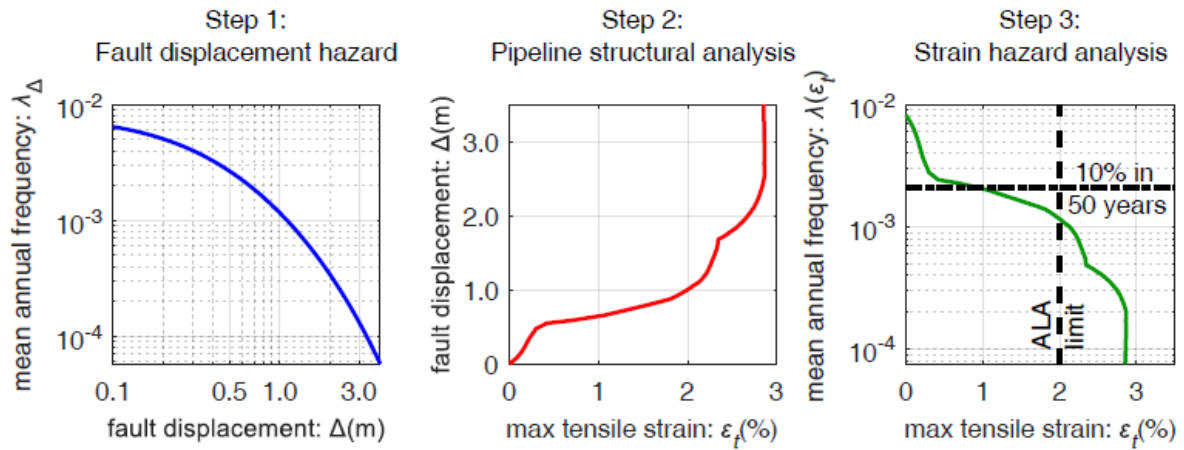
167 where ν_F is the recurrence rate of the fault, namely the annual average number of earthquake
 168 events with magnitude above a minimum one of engineering significance (M_{min}),
 169 $P(\Delta > \delta | m_i)$ is the probability that the fault displacement Δ exceeds a defined value δ at the
 170 crossing site, and finally $P_M(m_i)$ is the probability of magnitude M falling within the i -th bin,
 171 following an appropriate discretization of magnitude values between M_{min} and M_{max} .
 172 Parameters considered in the estimation of the probability $P(\Delta > \delta | m_i)$ are the fault
 173 mechanism, the fault length, the location of the crossing point on the fault trace, the maximum
 174 earthquake magnitude, and the probability of the rupture reaching the surface. This approach
 175 allows, also, the incorporation of aleatory and epistemic uncertainties. The former stem from
 176 the inherent variability of nature over time (e.g. earthquake magnitude, fault location) and may
 177 be handled via sampling. The latter originates from the inadequate understanding of nature (e.g.
 178 fault displacement prediction equations) and can be reduced in time with better observations
 179 while being typically handled for the time being via logic trees (Abrahamson and Bommer
 180 2005).

181 The pipe design within a performance-based framework requires the definition of the
 182 appropriate intensity measure (IM), a quantity that indicates the level/magnitude of the
 183 earthquake and acts as an interface between seismology and structural engineering (Bakalis
 184 and Vamvatsikos 2018). In the case of ground shaking, an IM selection procedure is required
 185 for buried pipelines subjected to earthquake excitation (Tsinidis et al. 2020). Contrarily, it is
 186 pretty straightforward that the fault displacement is the appropriate scalar IM in the case of
 187 fault crossing, provided that duration-dependent failure modes, such as low-cycle fatigue, are
 188 not examined. Strictly speaking, the fault offset is generally three-dimensional (Fig. 3).
 189 Typically, the principal fault displacement component is much higher than the other two and

190 determines the faulting mechanism. If the fault parallel component is the principal one, the
191 faulting mechanism is strike-slip. In cases the fault perpendicular component is the principal
192 one, the fault mechanism is normal or reverse (dip slip). Due to the lack of published data on
193 the relationship between the fault displacement components, only reasonable engineering
194 assumptions can be made (e.g. Melissianos et al. 2017b), leading to the introduction of a vector
195 IM (Baker 2007). A brief overview of the performance assessment of a buried pipeline
196 subjected to fault rupture is schematically shown in Fig. 4. The methodology comprises of three
197 steps: (1) fault displacement hazard analysis, (2) pipeline structural analysis, and (3) pipeline
198 strain hazard analysis. The outcome is the strain hazard curve that allows the evaluation of pipe
199 performance in strain terms with respect to code-based strain limits and the design return
200 period. The strain hazard curve is obtained via the convolution of results from steps (1) and
201 (2). The uncertainties associated with demand (strains obtained from structural analysis) and
202 capacity (code-based strain limits) have not been considered here for the sake of simplicity.



203
204 Fig. 3. Three-dimensional fault displacement, fault displacement components: Δ_1 fault-parallel,
205 Δ_2 vertical, and Δ_3 perpendicular to the fault plane.

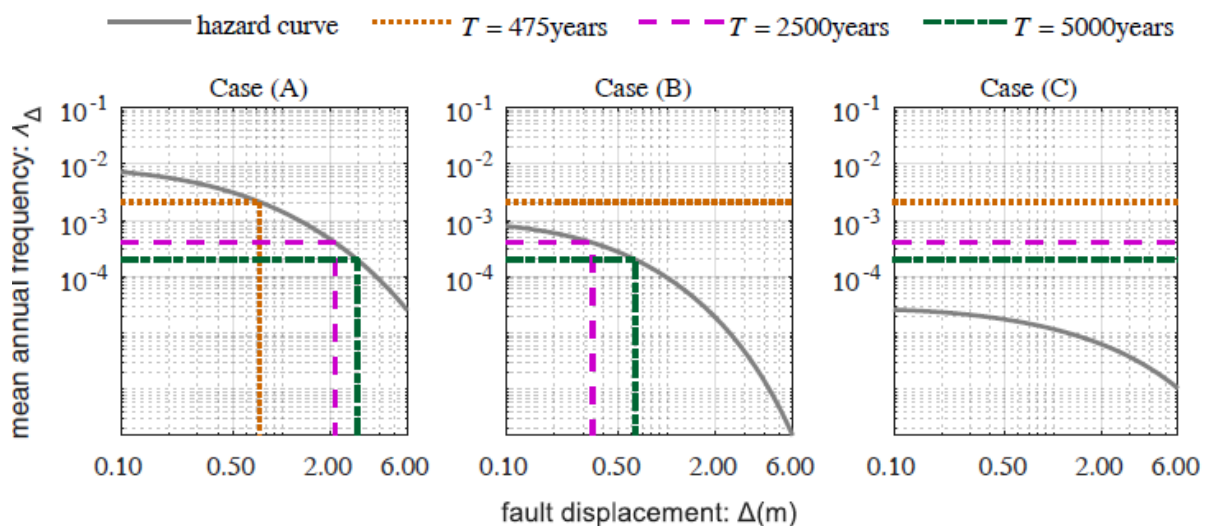


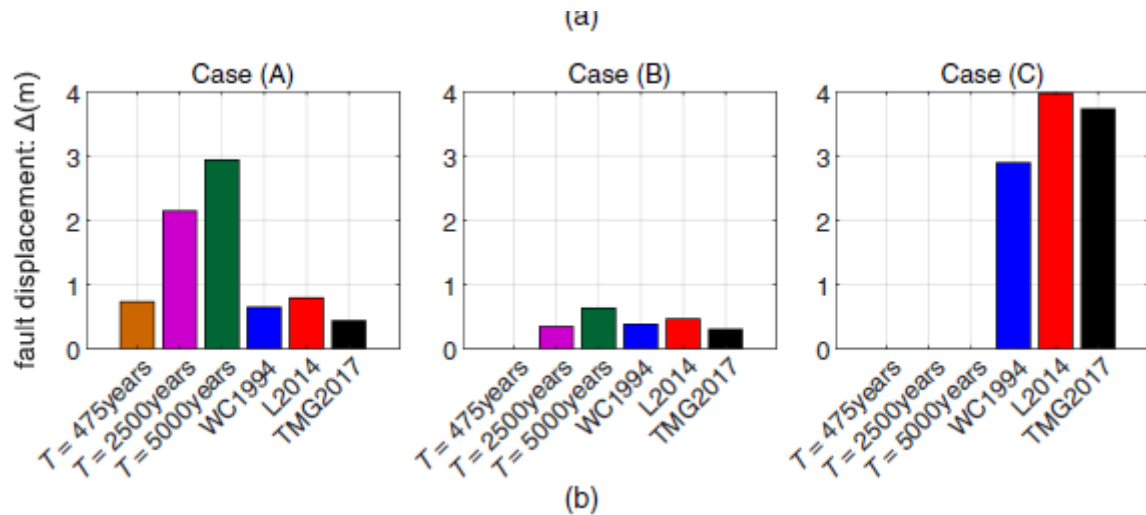
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207 Fig. 4. Performance-based assessment of buried pipeline at fault crossing: A schematic
 208 illustration [adapted from Melissianos et al. (2017c)]

209 Alternative estimations of the design fault displacement using empirical relations and
 210 full probabilistic analysis are discussed via an illustrative example of three pipeline–fault
 211 crossings in Europe. The fault properties have been obtained from the fault database created
 212 for the development of the 2020 European Seismic Hazard Model (Danciu et al. 2019) within
 213 the EU-funded research project SERA (Giardini et al. 2017). A buried pipeline is considered
 214 to be intercepted by a fault with a length L_F and recurrence rate ν_F , which is the average annual
 215 number of earthquake events above magnitude $M_{min} = 5.5$ of engineering significance. The
 216 pipeline is assumed to cross the fault at the middle of its length. The examined cases are: (A)
 217 a highly active normal fault close to Athens, capital of Greece with $L_F = 38.14\text{km}$ and
 218 recurrence rate $\nu_F = 0.0425\text{year}^{-1}$, (B) a very short strike-slip fault in the northern part of
 219 Turkey with length $L_F = 22.88\text{km}$ and $\nu_F = 0.0042\text{year}^{-1}$, and (C): a very long strike-slip
 220 fault in northwest France with $L_F = 159.74\text{km}$ and low seismicity (recurrence rate) $\nu_F =$
 221 0.0008year^{-1} . For each one of the three cases, the (median average surface) fault displacement
 222 Δ is calculated based on the fault length via the relations of Table 2. Then, the displacement
 223 values that correspond to return periods (T) equal to 475years, 2000years, and 5000years are

224 calculated from the hazard curves that are computed using the methodology of Melissianos et
 225 al. (2017b, 2021). The obtained fault displacements from the hazard curves (upper row of Fig.
 226 5) are compared to the ones calculated using alternative fault scaling relations (lower row of
 227 Fig. 5). It is observed that the fault scaling relations should be used as an indication of the
 228 “expected” Δ based on the fault dimensions. Essentially, ignoring the fault productivity
 229 (recurrence rate) leads to an unknown safety level and conservatism. The fault in Case (A) has
 230 a high recurrence rate and thus the fault displacement values corresponding to return periods
 231 of 2,500 and 5,000 years are above 2 m, while the displacement values obtained from the
 232 scaling relations are roughly below 1 m because the fault length is small. Contrarily, the fault
 233 in Case (C) has a very low recurrence rate and the computed values are consequently low, while
 234 the displacements obtained from the scaling relations are excessive due to the length ($L_F =$
 235 159.74km). The fault in Case (B) is very short with a relatively low recurrence rate, leading
 236 to comparable values of fault displacement between the two approaches. All-in-all, one should
 237 be very careful when employing the empirical fault scaling relations, acknowledging the
 238 limitations and the potential inconsistency with the code-required design return period.





240

241 Fig. 5. Estimation of design fault displacement via empirical relations and probabilistic analysis

242 [WC1994: Wells and Coppersmith (1994), L2014: Leonard (2014), TMG2017: Thingbaijam

243 et al. (2017)].

244 Numerical modeling

245 The pipeline mechanical behavior due to faulting can be numerically evaluated by developing

246 a beam-type or a continuum model (Xu et al. 2021). In more detail:

- 247 • Beam-type model: The pipeline is meshed into beam-type elements, allowing the calculation
248 of stresses and strains at selected integration points on the cross-section and along the
249 elements. The surrounding soil is modeled with non-linear translational springs in four
250 directions: axial, transverse horizontal (lateral), vertical upwards, and vertical downwards.
251 This model is presented in ASCE Guidelines, ALA Guidelines, EN 1998-4, CSA Z662
252 (Table 1) and adopted by researchers for the assessment of pipeline behavior (eg. Joshi et
253 al. 2011; Melissianos and Gantes 2017; Trifonov 2018; Uckan et al. 2015). The beam-type
254 model is routinely applied for the design of pipeline projects via commercial software, such
255 as CAESAR II, AutoPIPE, and Rohr because the development is easy and fast.

256 • Continuum model: The pipeline is discretized into shell finite elements and the surrounding
257 soil into 3D-solid elements. The pipe–soil interaction is modeled with contact elements.
258 Starting from the innovative work of Vazouras et al. (2010), the continuum model is
259 employed by researchers to gain in-depth knowledge of the pipe behavior (e.g. Gawande et
260 al. 2019; Rahman and Taniyama 2015; Trifonov 2015; Vazouras et al. 2015) and to calibrate
261 numerical models based on experimental results (Fadaee et al. 2020; Ni et al. 2018; Robert
262 et al. 2016; Rofooei et al. 2018; Sarvanis et al. 2018; Tsatsis et al. 2019; Wijewickreme et
263 al. 2017). It should be noted that continuum models are typically employed to estimate the
264 pipe-soil interaction in a more accurate manner than the simplified beam-type models
265 because the pipe-soil interface is modeled with contact elements that allow separation and
266 sliding of the soil around the pipe. This approach, even though still subjected to several
267 approximations, is the best available numerical approach to capture the physics of the
268 problem, provided that the model is carefully developed and an appropriate soil material has
269 been considered. Also, full-scale testing of buried pipelines subjected to faulting
270 is undoubtedly difficult, costly, and time-consuming. Thus, limited experimental results are
271 available and consequently, the validation of continuum numerical models is admittedly
272 restrictive. Still, comparison to the few available experimental results increases the
273 reliability of numerical results. Furthermore, non-linear springs might be used as an
274 alternative for soil modeling instead of 3D-solid elements (e.g. Gantes and Bouckovalas
275 2013; Karamitros et al. 2007; Kouretzis et al. 2011; Talebi and Kiyono 2020; Xu and Lin
276 2017; Zhang et al. 2017). In such a case, special attention is required regarding the mesh
277 density because, in the unlikely case of a coarse mesh, unrealistic local forces from springs
278 acting on the shell might alter the local buckling estimations. The continuum model can be
279 developed using general-purpose FEM software, such as Abaqus, ADINA, LS-DYNA, and
280 ANSYS.

281 The selection of the appropriate modeling technique depends primarily on the required
282 accuracy of the numerical predictions in terms of pipe local buckling and cross-section
283 ovalization and secondary on the available computational resources and the experience of the
284 pipeline engineer in advanced numerical modeling. One should bear in mind that a beam-type
285 model can offer a reasonable estimation of the pipe–soil interaction until the local buckling
286 occurrence (Karamanos et al. 2021). After that limit state, more advanced numerical models
287 are required in terms of modeling the pipe with shell elements. The main advantages and
288 disadvantages of each numerical approach are listed in Table 3.

289 ***Beam-type model***

290 The critical aspect of pipe modeling as a beam resting on foundation (Winkler approach) is the
291 force-displacement relationship of each soil spring, i.e. the numerical representation of the
292 pipe–soil interaction. The industry-standard ALA Guidelines are typically used to estimate the
293 soil spring properties. The basic inherent assumptions are that (1) the pipe is buried in uniform,
294 dry, or fully saturated backfill, (2) the soil mechanical properties are independent of the stress
295 level, and (3) the soil failure can freely develop, while the burial depth until the results are
296 reliable is unknown (Kouretzis and Wu 2021).

297 Axial soil springs model the pipeline–soil friction with properties depending on backfill
298 soil material and pipe coating. The maximum axial soil force per unit length can be estimated
299 after ALA Guidelines based on geotechnical approaches that are used to model the force
300 transfer on axially loaded piles (Singhal 1980):

$$T_u = \pi Dac + \pi DH\bar{\gamma} \frac{1 + K_0}{2} \tan \delta \quad (4)$$

301 where D is the pipe diameter, a is the adhesion factor, c is the soil cohesion representative of
302 the soil backfill, H is the depth to the pipe centerline, $\bar{\gamma}$ is the effective unit weight of the soil,
303 K_0 is the coefficient of pressure at rest, and δ is the interface angle of friction for pipe and soil

304 that depends on the internal friction angle of the soil and the type of pipe coating. The maximum
305 displacement depends on the soil type. Wijewickreme et al. (2009) performed full-scale axial
306 pull-out tests of pipes buried in dry sand. The axial load was found to be similar to the one
307 predicted by ASCE Guidelines for loose sand, while it was found much higher for dense sand.
308 In cases of dilatant sandy soil (e.g. in compacted trenches), the authors noted that the use of
309 existing code relations may lead to the underestimation of soil loads. Meidani et al. (2017)
310 performed axial pull-out tests and discrete element analyses on pipes embedded in dense sand.
311 It was shown that the axial resistance was higher than the one predicted by ASCE Guidelines
312 because the dense sandy soil surrounding the pipe was not at rest, implying that the actual
313 pressure coefficient should be higher than K_0 for relatively dense sand material. Sarvanis et al.
314 (2018) performed axial pull-out full-scale tests and proposed an updated version of the ASCE
315 Guidelines to take consider the stress increase at the pipe interface. This increase is related to
316 the fact that the sand backfill might not freely expand due to confined shear conditions caused
317 by sand dilatancy. Recently, Marino and Osouli (2020) performed an experimental campaign
318 on the slip resistance of coal tar-coated buried pipes in clay and sand. The authors concluded
319 that more analysis and testing is required on the coating (reduction) factors due to their
320 dependency on soil type and the unsaturated soil strength should be considered if that is the
321 soil case. To sum up, it should be considered for the design that if the backfill is purposely
322 compacted (e.g. under road crossings), considering the “at-rest” lateral earth pressure
323 coefficient K_0 would result in decreased axial loads on the displaced pipe (potentially unsafe
324 condition) and consequently to lower strains within the zone of ground deformation. In brief,
325 in the case of dense sand backfill, the axial soil resistance would be higher than the one
326 predicted by codes. However, more experimental and numerical analyses are required to
327 formulate an updated expression for axial soil resistance for code implementation.

328 Transverse horizontal (lateral) springs model the soil resistance to pipe lateral movement
329 in the trench. This mechanism has been considered to be similar to one of vertical anchor plates
330 or horizontal moving foundations by passive earth pressure (Trautmann and O'Rourke 1983).
331 This approach assumes that the soil failure could be fully developed or in other words the trench
332 geometry and the native soil (outside the trench) do not affect the development of soil failure
333 surfaces. A qualitative statement is only included in ALA Guidelines, stating that the trench
334 dimensions shall be "adequate", i.e. the trench and the surrounding native soil can be regarded
335 as "infinite" free-field conditions. The maximum lateral soil force may be estimated after ALA
336 Guidelines as:

$$P_u = N_{ch}cD + N_{qh}\bar{\gamma}HD \quad (5)$$

337 where N_{ch} is the horizontal bearing capacity factor for clay, N_{qh} is the horizontal bearing
338 capacity factor. The displacement corresponding at P_u is a linear function of burial depth and
339 pipe diameter. Regarding the effect of trench geometry in the development of the soil failure
340 surfaces, Kouretzis et al. (2013) carried out a numerical study to investigate the shape and size
341 of the failure surface for pipes being laterally displaced in loose and medium-density sand
342 backfills. Chaloulos et al. (2015) employed 2D finite element modeling and examined laterally
343 displaced pipelines. The authors identified three failure mechanisms for loose to medium
344 density sand backfills concerning the burial depth to diameter ratio, namely shear failure, local
345 shear failure, and intermediate shear failure modes. Chaloulos et al. (2017) extended their
346 previous work and developed a straightforward analytical methodology for the computation of
347 the soil pressure applied on a laterally displaced pipeline concerning the trench geometry and
348 shape (width and wall inclination). A set of modification factors was developed for design in
349 case the trench is excavated in stiff soils and rocks. Then, pipeline–soil lateral interaction
350 remains a topic under investigation via experimental tests and detailed numerical models.

351 Notable experimental and numerical studies have been conducted during the last decade, which
352 highlight the need for code revision. Tian and Cassidy (2011) developed a pipe–soil interaction
353 model for a pipe under horizontal and vertical loading and introduced a radial hardening term
354 that was found to be necessary for pipes being horizontally displaced twice their diameter due
355 to accumulated soil berm. Daiyan et al. (2011) conducted centrifuge tests and FE analysis,
356 concluding that the lateral soil force depends on the movement angle of the pipe in the trench
357 to soil friction angle and burial depth. Jung et al. (2013) performed a combination of physical
358 tests and FE analysis showing that the dimensionless maximum lateral force mobilized by
359 large-diameter pipes buried in dense sand is decreased at low depth to diameter ratios. Roy et
360 al. (2016) employed FE modeling to analyze the pipe–soil interaction using a Modified Mohr-
361 Coulomb soil model, aiming to replicate experimental results and to investigate the effects of
362 pipe diameter, burial depth, and soil properties. Robert et al. (2016) carried out experimental
363 tests and numerical analyses and found that if a pipeline is buried in unsaturated sand, then soil
364 lateral resistance is higher than anticipated and should be considered in the analyses. Ono et al.
365 (2017) conducted lateral loading experiments in pipes buried in sand focusing on the influence
366 of the initial effective stress. The authors developed a force–displacement relationship to
367 account for the variation of soil unit weight concerning pore water pressure, burial depth, pipe
368 diameter and length. Robert (2017) developed a modified Mohr-Coulomb model to simulate
369 the behavior of pipes in unsaturated soils because soil friction affects the lateral loads imposed
370 on pipes. Roy et al. (2018a) performed numerical analyses of laterally loaded pipes in dense
371 sand and revealed that the peak and residual resistances increase with the embedment ratio until
372 a critical value that depends on the pipe diameter. Nguyen and Asimaki (2018) proposed a
373 modified uniaxial Bouc-Wen model to evaluate the force–displacement backbone curve for the
374 lateral interaction of pipe–sandy soil. Chakraborty (2018) numerically examined pipes
375 embedded in a cohesive soil and showed that the lateral soil capacity decreases with the

376 decrease of pipe burial depth to diameter ratio and soil friction angle. Tahamouli Roudsari et
377 al. (2019) experimentally investigated pipes under strike-slip faulting using a shear box and
378 found that the maximum lateral soil force was 70% higher than the one estimated after ASCE
379 Guidelines and close to the one estimated after ALA Guidelines for steel pipes. In the case of
380 HDPE pipe, the force was 80% lower than ASCE Guidelines, demonstrating that the code
381 expression should be adjusted to incorporate the effect of pipe material. Recently, Wu et al.
382 (2020) carried out tests to investigate the transition from a shallow to a deep failure mechanism
383 concerning pipe burial depth and friction angle. Ashrafy et al. (2020) proposed a modification
384 of ASCE Guidelines and ALA Guidelines equations for lateral soil resistance for thick pipes
385 buried in dense sand and subjected to strike-slip faulting. Dilrukshi and Wijewickreme (2020)
386 examined the influence of the particle size of soil backfill material with respect to the pipe size
387 and formed a relation for the peak lateral force considering burial depth to diameter and pipe
388 diameter to soil particle size ratios. The authors formulated an expression for practical
389 application in pipe design. Finally, Ansari et al. (2021) performed small-scale tests in a soil
390 chamber to investigate the soil resistance to pipe lateral movement in loose to very dense dry
391 sand. The authors concluded that the existing equations for dense sand backfill underestimate
392 the lateral soil resistance. Summarizing, the following main remarks should be considered
393 regarding the lateral soil force–displacement relationship:

- 394 • Code relations assume “free-field” conditions or in other words, it is assumed that the failure
395 surface is developed within the trench. If these assumptions are not satisfied, then
396 appropriate modification factors should be applied to the code relations. The analytical
397 approach of Chaloulos et al. (2017) to compute the trench size and shape effect could be
398 practically employed for pipeline design. The methodology is outlined in Table 4.
- 399 • The lateral soil resistance of unsaturated sand is higher than expected. Code relations should
400 be modified for pipe material other than steel. There are preliminary findings that the

401 movement angle of the pipe in the trench affects the soil force. Finally, soil resistance to
402 pipe lateral movement is underestimated in dense sandy backfill.

403 Vertical upward springs model the uplift soil resistance due to the upward movement
404 of the pipe in the trench. The maximum soil force corresponds to the weight of an inverted
405 prism of soil above the pipeline top (O'Rourke and Liu 2012). The maximum vertical uplift
406 soil force may be estimated after ALA Guidelines:

$$Q_u = N_{cv}cD + N_{qv}\bar{\gamma}HD \quad (6)$$

407 where N_{cv} is the vertical uplift factor for clay, N_{qv} is the vertical uplift factor for sand. The
408 corresponding displacement at Q_u is a linear function of burial depth and depends on the pipe
409 diameter and soil type. There is significant research effort on the uplift soil mechanisms of
410 buried offshore pipelines in light of preventing thermal upheaval buckling. Contrarily, the
411 number of studies for onshore pipelines during the last decade is limited. Jung et al. (2013a)
412 performed 2D FE analyses to examine the uplift pipe–sandy soil interaction and developed
413 hyperbolic and bilinear relations for the uplift force–displacement curves taking into account
414 the density and the burial depth to diameter ratio. Chakraborty and Kumar (2014) performed
415 FE analyses and examined the variation of friction angle in the soil domain concerning the
416 sandy soil type, the burial depth, and the pipe diameter. Robert and Thusyanthan (2018)
417 examined the uplift resistance of buried pipes in partially saturated sand because the latter is
418 not considered in the existing expressions. The authors performed tests in a soil chamber and
419 found that the uplift resistance of deeply buried pipes in denser soils is lower than the one
420 obtained from ASCE Guidelines. A model was proposed to evaluate the uplift resistance of
421 small diameter pipes in partially saturated sand. Wijewickreme et al. (2017) executed full-scale
422 experiments modeling pipes under reverse faulting to evaluate the soil mobilization due to the
423 pipe upward movement in the trench with respect to fault dip angle and soil friction angle. The

424 authors proposed modifications to existing expressions (soil springs) regarding the distance
425 from the fault trace. Roy et al. (2018) proposed a modified Mohr-Coulomb law for design
426 applications regarding the uplift resistance by considering the effect of burial depth on the
427 developed failure surfaces, the inversely proportional relation between displacement and burial
428 depth, and the decrease of uplift resistance at large displacements with the increase of upward
429 movement. Wu et al. (2020b) performed a series of physical tests and concluded that the
430 existing relations should be used with caution even for shallowly buried pipes because the
431 failure mechanism is almost independent of sand density. Very recently, Cugnetto et al. (2021)
432 employed FE modeling of pipes buried in dry sand, using advanced soil material laws. The
433 authors found the simplified linear elastic-perfectly plastic Mohr-Coulomb material law can be
434 used to safely estimate the upward soil resistance. The code expressions yield conservative
435 results (overprediction) for deeply buried pipelines, while for shallowly buried ones, the
436 corresponding results are acceptable. To summarize, current expressions yield conservative
437 results due to the interaction between the pipe burial depth, the pipe upward movement, and
438 the uplift mobilized soil force.

439 Pipe vertical downward movement results in the development of forces at the pipe–soil
440 interface that corresponds to the vertical bearing capacity of a footing (O’Rourke and Liu
441 2012). The maximum vertical downward soil force after ALA Guidelines is:

$$Q_d = N_c cD + N_q \bar{\gamma} HD + N_\gamma \gamma D^2 / 2 \quad (7)$$

442 where N_c , N_q , N_γ are bearing capacity factors and γ is the total unit weight of soil. The
443 maximum displacement is a linear function of the pipe diameter and depends on soil cohesion.
444 Xie et al. (2013) numerically examined pipes subjected to normal faulting and found that the
445 bearing capacity was 1/8 the value estimated after ASCE Guidelines and the bearing capacity
446 varied along the pipe, with the lower value found close to the fault plane. An update of code

447 expressions was provided. Kouretzis et al. (2014) proposed an updated set of expressions for
448 computing the downward soil resistance because the existing ones in ALA Guidelines can
449 significantly underestimate or overestimate the soil resistance. The authors gave practical
450 suggestions regarding the trench dimensions to avoid the interaction between the soil failure
451 surface and the potentially stiffer native soil. O'Rourke et al. (2016) examined the response of
452 pipelines subjected to vertical ground movement, reviewed measured stress of pipe in full-scale
453 tests, and discussed the pipeline–soil interaction under normal faulting taking into account the
454 coupling of frictional forces and soil reaction forces normal to the pipeline axis. It was
455 concluded that the maximum downforce is about 1/3 the one derived from existing equations.
456 Jung et al. (2016) carried out full-scale tests of pipes under faulting and numerical analyses to
457 examine soil restrains. The maximum downward force was found to be 1/3 of the
458 corresponding one after existing equations, rendering code provisions as overly conservative.
459 Also, the pipe diameter was a parameter affecting soil force for constant burial depth, i.e.
460 increasing the diameter led to increasing soil stresses. Recently, Qin et al. (2019) numerically
461 examined rigid pipes embedded in granular soil and subjected to downward movement. The
462 authors found that the code expressions overestimate the bearing capacity and proposed a new
463 force–displacement relation based on the local shear failure theory, compared to the general
464 shear failure theory adopted by codes. Cugnetto et al. (2021) focused on the investigation of
465 the downward soil resistance of buried pipelines embedded in dry sand and subjected to vertical
466 fault movement. The continuum FE model was employed and it was noticed that simplified
467 linear elastic-perfectly plastic Mohr-Coulomb material law can be used to safely estimate the
468 downward soil resistance. Contrarily, it was verified that the code expressions are over-
469 conservative. The authors built a statistical model (fitting expression) based on the numerical
470 results for predicting the soil downward resistance force with respect to the pipeline length, the
471 embedment ratio, and the soil properties for pipes buried in noncohesive, dense to loose, and

472 dry homogenous sand material. Summarizing, it is concluded that the code relations for
473 estimating the soil downward resistance provide overestimations, leading to conservative pipe
474 design and highlighting the need for revision. Regarding the trench geometry, as stated by
475 Kouretzis et al. (2014), the higher the burial depth and the soil friction angle, the larger the
476 cross-section area of the trench should be in terms of both width and wall inclination. This is to
477 allow the formation of the soil failure modes. Otherwise, a cost-benefit analysis is required to
478 compare the cost of additional excavations and the implementation of mitigation measures.

479 Finally, the recent study of Kouretzis and Wu (2021) presents a comprehensive and
480 complete set of recommendations for the estimation of the lateral and vertical soil spring
481 properties based on a set of new experimental results by Ansari et al. (2018). The updated
482 expressions take into account the dependency of sand properties on the confining stress levels.
483 The response of typical buried steel pipelines in terms of strains was evaluated using the ALA
484 Guidelines and the updated expressions for soil spring properties. It was found that in the
485 former case, pipe strains were significantly higher, demonstrating the conservatism of ALA
486 estimations. The proposed set of expressions is listed in

487

488 **Table 5** for potential design application.

489 ***Continuum model***

490 The pipe–fault crossing analysis with a continuum numerical model allows among others to
491 assess the pipe local instability (shell buckling) and cross-section ovalization, model the trench
492 cross-section shape, and use advanced soil material laws. However, the model length is limited
493 and the pipe segments beyond the curved length and until the anchor points should be
494 represented using a simplified approach. These three aspects are discussed in detail
495 subsequently.

496 Trench geometry is typically not considered in the analysis assuming that the pipe is buried
497 in homogeneous soil and consequently soil failure surfaces can freely develop. Still, if the
498 trench is excavated in stiff or rocky soil, then modeling the trench geometry is required, as
499 discussed by Trifonov (2015) and Cheng et al. (2019).

500 An elastic–perfectly plastic Mohr-Coulomb law is typically employed for soil modeling
501 (e.g. Soveiti and Mosalmani 2020; Vazouras et al. 2015). Trifonov (2015) adopted a Drucker-
502 Prager criterion to avoid the computational difficulties associated with corners at the yield
503 surface of the Mohr-Coulomb law. Experimental results could, also, be used for the
504 development of custom-made laws (e.g. Dey et al. 2020). Recently, Robert et al. (2020)
505 developed codes for practical application in commercial software to model the pipeline–soil
506 interaction, focusing on dry and unsaturated soils.

507 Modeling a very long segment of the pipeline is not viable because excessive
508 computational resources are required. Thus, Vazouras et al. (2010) suggested a modeling length
509 equal to 60 times the pipe diameter. The boundary conditions at the pipeline ends play a non-
510 negligible role and affect the pipeline behavior, a task that can be handled via alternative
511 approaches:

512 • The pipeline beyond the continuum model is replaced by an equivalent spring to consider
513 the axial deformation of the pipe. Liu et al. (2004) discussed this topic and developed an
514 equivalent spring via a simplified approach. Vazouras et al. (2015) developed an equivalent
515 spring for an infinitely long pipeline and a finite-length pipeline by analyzing separately the
516 sliding and the non-sliding segment (inelastic and elastic behavior at the pipe–soil interface,
517 respectively) of the pipeline. Zhang et al. (2016b) update the model of Liu et al. (2004),
518 introducing discrete cases along the straight segment of the unanchored length regarding
519 pipe steel yielding (elastic and inelastic pipe behavior) and yielding of axial springs.
520 Recently, Banushi and Squeglia (2018) provided an advanced methodology for estimating
521 the force–displacement relation of the equivalent springs for pipelines subjected to strike-
522 slip faulting, considering the pipeline operating temperature and the internal pressure and
523 providing different relations for tension and compression. To summarize, the available
524 methodologies for computing the properties of the equivalent spring require detailed
525 calculations for each segment of the force–displacement relationship. The required pipe–
526 soil interaction parameters can be obtained from either experimental tests or additional
527 advanced numerical analyses of pipe pull-out loading. The soil stiffness and the shear
528 strength at the pipe–soil interface are required in the methodology of Vazouras et al. (2015).
529 The properties of the equivalent spring after the methodology of Banushi and Squeglia
530 (2018) are a function of soil and pipeline nonlinear properties, elastic rigidity of pipe–soil
531 friction interaction, internal pressure and temperature variation, pipeline unanchored length,
532 and pipe cross-section area. Contrarily, the methodology of Zhang et al. (2016b) seems
533 simpler, requiring the pipe cross-section area, the steel modulus of elasticity, and the
534 maximum axial soil spring force along with the corresponding yielding displacement (both
535 might be obtained from ALA Guidelines). It is noted that each methodology was founded
536 on different assumptions and consequently a direct comparison is not viable.

537 • The pipeline segment beyond the continuum model is modeled with the beam-type model
538 (e.g. Gantes and Bouckovalas 2013; Zeng et al. 2019), connecting the two parts (continuum
539 and beam-type model) via rigid links.

540 **Protection Measures**

541 The protection of buried pipelines at fault crossing results from a blend of regulatory provisions
542 (Table 1), engineering judgment (e.g. Darigo et al. 2008; Keaton and Honegger 2008), and
543 requirements of the pipeline owner. The general regulatory recommendations are (1) pipe
544 rerouting to avoid environmentally sensitive and populated areas, (2) pipe orientation (selection
545 of pipe–fault crossing angle) that results in pipe tension, rather than compression, (3)
546 minimization of burial depth to reduce soil restrains on the pipe during movement in the trench,
547 (4) avoidance of sharp bends in the crossing area that might act as anchor points (Nair et al.
548 2019), and (5) trench backfilling with appropriate soil material over a distance of 50m on each
549 side of the fault trace. These recommendations stand as the “first line of defense” against the
550 consequences of faulting but might not be sufficient enough to ensure the pipe safety or not
551 applicable due to environmental restrictions, the presence of physical obstacles, and regulatory
552 restrictions. Thus, specific seismic countermeasures are typically required, the selection of
553 which is based on a cost-benefit analysis using appropriate variables, such as procurement and
554 installation cost, pipe–fault crossing geometry, pipe owner specifications, and regulatory
555 provisions.

556 The protection of buried pipes at fault crossings might be seen as a trivial or very broad
557 issue that is handled on a case-by-case basis. Nevertheless, a comprehensive and critical review
558 of the international engineering practice remains useful for designers and pipe owners. At the
559 same, it should be noted that the literature on pipeline protection measures is very limited
560 compared to pipeline mechanical behavior studies. Qualitative discussions on protection
561 measures are offered by Nyman et al. (2008), O’Rourke and Liu (2012), and Karamanos et al.

562 (2017). Quantitative comparisons of measures are presented by Gantes and Melissianos (2016),
563 Melissianos et al. (2017c), Melissianos and Gantes (2019), and Valsamis et al. (2020). In these
564 studies, the authors have grouped the measures into three categories, based on the mechanism
565 employed to achieve pipe strain reduction: pipe strengthening, soil friction reduction, and
566 complex measures.

567 *Types of protection measures*

568 Pipe strengthening can be achieved by:

- 569 • Steel grade upgrade to improve strength (Gantes and Bouckovalas 2013; Karamanos et al.
570 2017),
- 571 • Wall thickness increase to improve pipe cross-section stiffness (Gantes and Bouckovalas
572 2013; Karamanos et al. 2017).
- 573 • Pipe wrapping with composite wraps to increase strength (Mokhtari and Alavi Nia 2015;
574 Trifonov and Cherniy 2014, 2016).

575 The reduction of the friction developed on the pipe-soil interface contributes to the reduction
576 of pipe strains. This could be accomplished by the:

- 577 • Trench backfilling with tire-derived aggregate, which is a compressible material (Ni et al.
578 2018; Sim et al. 2012).
- 579 • Use of geotextile-lined pipeline trenches (Gantes and Bouckovalas 2013) that have a
580 marginal effect on pipelines subjected to strike-slip faulting (Monroy-Concha et al. 2012).
- 581 • Trench backfilling with loose granular soil, for example, pumice (Gantes and Bouckovalas
582 2013; Valsamis et al. 2020).
- 583 • Excavation of a wider trench for the pipeline to “freely” move in the trench (Gantes and
584 Bouckovalas 2013).

585 • Pipe isolation from ground displacements by placing the pipeline within concrete culverts
586 and without backfilling material in the case of strike-slip faulting (Gantes and Bouckovalas
587 2013; Tsai et al. 2015; Valsamis et al. 2020).

588 • Partial replacement of soil backfill with EPS geofoam blocks (Azizian et al. 2020; Bartlett
589 et al. 2015; Beju and Mandal 2017; Choo et al. 2007; D.G. Honegger Consulting SSD Inc.
590 2009; Rasouli and Fatahi 2020).

591 Other measures that have been proposed by scholars or applied on a case-specific basis and can
592 be classified neither as pipe strengthening nor as friction reduction are listed below:

593 • Zhang et al. (2016) examined a protective device that aims at reducing the potential of local
594 buckling by applying external hydrostatic pressure to the pipeline at critical predefined
595 locations.

596 • Besstrashnov and Strom (2011) proposed a pipe route changing with a very high radius bend
597 to allow unrestrained pipe deformation.

598 • Melissianos et al. (2016, 2017a), Valsamis et al. (2020), and Valsamis and Bouckovalas
599 (2020) have investigated the use of flexible joints as a novel design approach. Flexible joints
600 are introduced in the pipeline at the fault vicinity to “absorb” pipe deformation and render
601 pipe segments virtually undeformed and consequently unstrained. The structural system of
602 the pipeline is transformed from continuous to segmented because the joints “act” as internal
603 hinged. The commercial bellow-type joints are welded between pipe segments, thus
604 ensuring pipe continuity and excluding the risk of separation, which is a typical failure mode
605 of low-pressure segmented pipes. Experimental, analytical, and numerical studies carried
606 out by the authors have revealed that it is a very promising solution especially for pipes
607 being subjected to significant fault offset displacement.

608 • Hart et al. (2004) designed a case-specific pipeline–fault crossing consisting of a pipe offset
609 made of four cold high-radius bends.

610 • Vazouras and Karamanos (2017) investigated the potential use of field bends as a mitigation
611 measure to relieve pipe strains under very specific conditions, taking advantage of bends’
612 flexibility.

613 • Hasegawa et al. (2014) proposed the creation of a predefined buckling pattern that consists
614 of localized deformation of the pipe wall at specified predefined locations aiming at
615 controlling the pipe local deformation (Wham et al. 2019).

616 Finally, if no measure is efficient enough, the pipe might be elevated above the ground, a
617 solution that has been applied successfully at the Trans-Alaska – Denali Fault crossing
618 (Honegger et al. 2004).

619 The comparison of “conventional” measures presented by Gantes and Melissianos
620 2016; Melissianos et al. 2017c; Melissianos and Gantes 2019; Valsamis et al. 2020 yields the
621 following results for practical consideration:

622 • Pipe strengthening measures (wall thickness increase and steel grade upgrade) are
623 economically acceptable only for low to very low fault displacement ($\Delta < 1.0D$).

624 • Trench backfilling with fine-graded soil material is, in general, an efficient measure for
625 medium to high fault displacement ($1.0D < \Delta < 3.0D$).

626 • Pipe placement within culverts is a very expensive but efficient measure for very high strike-
627 slip fault displacement ($\Delta > 3.0D$).

628 • Pipe–fault crossing angle and fault dip angle are predominant parameters affecting the
629 effectiveness of protection measures.

630 *Selection criteria*

631 The selection of the appropriate protection measure is based on a set of criteria given the current
632 legislation and the pipe owner’s specifications. The categorical criteria set by Valsamis et al.
633 (2020) are adopted to group the parameters that drive the selection and formulate a set of

634 preliminary selection criteria (Table 6), which should be considered under the following
635 remarks:

- 636 • Protection measures are applied along the entire fault trace uncertainty length, thus affecting
637 the cost-related criteria.
- 638 • Weight factors should be applied if necessary, depending on the case at hand. For example,
639 if the crossing is located at a remote mountainous site, the transportation and installation
640 costs might be very high.
- 641 • More than one protection measure might be selected to satisfy the design objectives.

642 Five protection measures, namely wall thickness increase (pipe strengthening), pipe
643 placement within culverts and backfilling with pumice (soil friction reduction), and
644 introduction of flexible joints and route changing with high radius bends (complex) are
645 indicatively examined using the selection criteria of Table 6. The compliance of each measure
646 to every criterion is presented in Table 7, demonstrating that the selection process is a multi-
647 level cost-benefit analysis. Regarding the criterion “1.4 Requirement for sophisticated
648 analysis” of Table 6, brief practical guidelines for the numerical modeling of alternative
649 protection measures are offered in Table 8.

650 **Conclusions**

651 The performance of onshore buried steel fuel pipelines at fault crossings has been studied
652 extensively during the past years. Still, some aspects require the attention of the scientific
653 community. The state-of-the-art review presented has critically examined three topics: the
654 estimation of the design fault displacement, the critical numerical modeling aspects to be
655 considered in the design, and the selection of pipe protection measures. The main findings are
656 summarized as follows:

- 657 • Empirical fault scaling relations should be used as an indication or a deterministic cap of
658 the design fault displacement. A full probabilistic Figure analysis is suitable for estimating
659 the design fault displacement and achieving a balance between safety and economy.
- 660 • In case a beam-type numerical model is developed for the analysis of the pipe–fault crossing,
661 attention should be paid to the force–displacement curves for the soil springs provided in
662 codes. The engineer should be aware of the code assumptions and restrictions because there
663 are cases in terms of soil properties, loading conditions, etc. that these curves lead to either
664 conservative and expensive or unsafe pipe design. There are recently published expressions
665 for the lateral, vertical upward, and vertical downward soil springs’ force–displacement
666 curves that could be used in the design.
- 667 • In case a continuum numerical model is developed for the pipe analysis, an available
668 methodology should be considered to replace the pipe segments between the curved length
669 and the anchor points.
- 670 • The relationship between backfill and the native soil properties, as well as the trench
671 geometry in the case of significant lateral movement of the pipe, should drive the decision
672 on considering or not the trench cross-section shape in the numerical model.
- 673 • A variety of conventional and case-specific protection measures is available, which aim at
674 pipe strain reduction via pipe strengthening, soil friction reduction, or more complex
675 mechanisms. The selection of a protection measure results from a cost-benefit analysis. A
676 set of preliminary selection criteria has been developed for practical application, where
677 partial weight factors should be applied based on engineering judgment and a case-by-case
678 basis.

679 **Data availability statement**

680 Some or all data, models, or code that support the findings of this study are available from the
681 corresponding author upon reasonable request.

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1171 **Tables**

1172 **Table 1.** List of codes, standards, and guidelines for the design/assessment of buried
 1173 pipelines at fault crossings.

Document	Publisher	Country
AS/NZS 2885.1:2018 Pipelines - Gas and liquid petroleum - Part 1: Design and construction	Council of Standards Australia / New Zealand Standards Approval Board (2018)	Australia / New Zealand
Z662:19 Oil and gas pipeline systems	Canadian Standards Association (2019)	Canada
EN 1998-4:2006 Eurocode 8 – Design of structures for earthquake resistance – Part 4: Silos, tanks and pipelines	European Committee for Standardization (2006)	European Union
IITK-GSDMA Guidelines for seismic design of buried pipelines	Indian Institute of Technology Kanpur (2007)	India
ISO 20074:2019 Petroleum and natural gas industry — Pipeline transportation systems — Geological hazard risk management for onshore pipeline	International Organization for Standardization (2019b)	International
NEN 3650-1 Requirements for pipeline systems – Part 1: General requirements	Royal Netherlands Standardization Institute (2020)	The Netherlands
Good Practice Guide Seismic screening assessment of UK onshore pipelines and associated installations	United Kingdom Onshore Pipeline Operators' Association (2019)	UK
ASCE Guidelines of the seismic design of oil and gas systems	American Society of Civil Engineers (1984)	USA
ALA Guidelines for the design of buried steel pipe	American Lifelines Alliance (2001)	USA
PRCI PR-268-134501-R01 Pipeline seismic design and assessment guideline	Pipeline Research Council International (Honegger 2017)	USA
ASME B31.8-2018 Guide for gas transmission and distribution piping systems	American Society of Mechanical Engineers (2018)	USA
PRCI PR-350-164501-R01 Guidance for assessing buried pipelines after a ground movement event	Pipeline Research Council International (Wang 2019)	USA

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1175 **Table 2.** Empirical fault scaling relations for $\Delta \sim f(L_F)$ for $L_F > 10\text{km}$ [Wells and
 1176 Coppersmith (1994): WC1994, Leonard (2014): L2014, Thingbaijam et al. (2017):
 1177 TMG2017].

Reference	Expression for median value (Δ in m, L_F in km)	Expression parameters (α/β)		
		normal	reverse	strike-slip
WC1994	$\log_{10}(\Delta) = \alpha + \beta \log_{10}(0.75L_F)$	-1.990 / 1.240	-0.600 / 0.310*	-1.700 / 1.040
L2014+ (INT)	$\log_{10}(\Delta_{sub}) = \alpha + \beta \log_{10}(1000L_F)$ $\Delta = \Delta_{sub}/1.32$	-3.799 / 0.833	-3.799 / 0.833	$10 \leq L_F \leq 40$ -3.844 / 0.833 $L_F > 40$ -2.310 / 0.500
L2014+ (SCR)	$\log_{10}(\Delta_{sub}) = \alpha + b \log_{10}(1000L_F)$ $\Delta = \Delta_{sub}/1.32$	-3.572 / 0.833	-3.572 / 0.833	$10 \leq L_F \leq 60$ -3.615 / 0.833 $L_F > 60$ -2.022 / 0.500
TMG2017	$\log_{10}(\Delta_{sub}) = \alpha + \beta \log_{10}(L_F)$ $\Delta = \Delta_{sub}/1.32$	-2.302 / 1.302	-1.456 / 0.975	-1.473 / 0.789

*WC1994 expressions for reverse fault mechanism are not significant at a 95% probability level

+Note on tectonic environment: Interplate (INT) refers to the plate boundaries, while Stable Continental Region (SCR) refers to midcontinental earthquakes.

1179 **Table 3.** Beam-type and continuum numerical models for pipeline–fault crossings:
 1180 Advantages and disadvantages.

Model aspects	Model	
	Beam-type	Continuum
Model detail	low	high
Development	easy	difficult
Requirement for experienced engineer	no	yes
Computing resources required	low	very high
Analysis time	a few minutes	a few hours
Potential significant convergence issues	no	yes
Appropriate for pipeline route with many bends	yes	no
Steel material modeling	detailed	detailed
Soil material modeling	simplified	very detailed
Trench geometry modeling	no	yes
Strain estimation	yes	yes
Cross-section ovalization (direct) assessment	no	yes
Local buckling and wrinkles (direct) assessment	no	yes

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Table 4. Analytical approach to compute the trench size and shape effect in pipeline design after Chaloulos et al. (2017).

Action
<p>1 The ultimate soil pressure and displacement for both the natural ground (p_{ult}^{gr} and y_{ult}^{gr}) and backfill soil (without trench effects) are computed ($p_{ult,inf}^{bf}$ and $y_{ult,inf}^{bf}$) after code provisions.</p>
<p>2 If $p_{ult}^{gr} < p_{ult,inf}^{bf}$, then trench effects are omitted and natural ground properties are adopted for computing the lateral soil springs. If $p_{ult}^{gr} > p_{ult,inf}^{bf}$, then proceed to the following steps.</p>
<p>3 The minimum required horizontal ($x_{cr} = ax_{max}$) and vertical ($d_{cr} = D$) distances of the displaced pipeline are computed for ignoring the trench effects, where:</p> $a = \begin{cases} 2.7 + 1.8 \tanh[0.6(H/D - 8.5)], & \text{for loose backfill sand} \\ 1.5 + 0.6 \tanh[0.6(H/D - 8.5)], & \text{for medium backfill sand} \end{cases}$ $\frac{x_{max}}{H} = 3.5e^{-0.27(\frac{H}{D})} \text{ or } \frac{x_{max}}{D} = \begin{cases} 3.0 + 0.10(H/D)^{C_1} & \text{for } H/D > A \\ 13.1 - 1.2(H/D) & \\ C_2 & \text{for } H/D > B \end{cases}$ <p>with</p> <p>$C_1 = 1.9, C_2 = 1.1, A = 6.0,$ and $B = 10.0$ for loose sand $\gamma_{dry} = 14.8kN/m^3$ $C_1 = 2.4, C_2 = 1.7, A = 4.8,$ and $B = 9.5$ for medium sand $\gamma_{dry} = 16.4kN/m^3$.</p>
<p>4 If $d < d_{cr}$, then for limited trench depth, the correction factors $I_{d,p}$ and $I_{d,y}$ are:</p> $\begin{cases} I_{d,p} = 1.1 \pm 0.1 \text{ and } I_{d,y} = 1.0 & \text{for loose sand and } H/D < 9.5 \\ I_{d,p} = 1.2 \pm 0.2 \text{ and } I_{d,y} = 1.2 & \text{for loose sand and } H/D \geq 9.5 \\ I_{d,p} = 1.0 \pm 0.1 \text{ and } I_{d,y} = 0.8 & \text{for medium sand and } H/D < 9.5 \\ I_{d,p} = 1.2 \pm 0.2 \text{ and } I_{d,y} = 1.0 & \text{for medium sand and } H/D \geq 9.5 \end{cases}$ <p>If $d \geq d_{cr}$, then the correction factors are $I_{d,p} = I_{d,y} = 1.0$.</p>
<p>5 If $x < x_{cr}$, then for limited trench width, the correction factors $I_{w,p}$ and $I_{w,y}$ are:</p> $I_{w,p} = (x/x_{cr})^{-I_{\theta,p}b_p} \geq 1.0 \text{ and } I_{w,y} = (x/x_{cr})^{-I_{\theta,y}b_y} \geq 1.0 \text{ with}$ $b_p = 1.1 - 0.6 \tanh[0.32(H/D - 3.2)]$ $I_{\theta,p} = 1 - 0.35\{1 - \tanh[0.32(H/D - 6.3)]\}\sqrt{\cos \theta}$ $b_y = \begin{cases} 0.55 - 0.55 \tanh[0.42(H/D - 4.2)] & \text{for loose sand} \\ 0.70 - 0.70 \tanh[0.35(H/D - 5.5)] & \text{for medium sand} \end{cases}$ $I_{\theta,y} = b_{y,\theta}/b_y = 1 + (I_{\theta,p} - 1)b_p/b_y$ <p>If $x \geq x_{cr}$, then the correction factors are $I_{w,p} = I_{w,y} = 1.0$.</p>
<p>6 The ultimate soil pressure (p_{ult}^{bf}) and displacement (y_{ult}^{bf}) for considering trench effect in sand backfill are: $p_{ult}^{bf} = I_{d,p}I_{d,y}p_{ult,inf}^{bf}$ and $y_{ult}^{bf} = I_{w,p}I_{w,y}y_{ult,inf}^{bf}$. The pipeline analysis is performed using the minimum ultimate soil pressure and associated ultimate displacement of natural soil and backfill sand.</p>

Notations: $\begin{cases} D & \text{pipe diameter} \\ H & \text{burial depth} \\ \theta & \text{trench inclination} \end{cases}$

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Table 5. Determination of spring properties after Kouretzis and Wu (2021) for design application of buried steel pipelines embedded in sand backfill and subjected to fault rupture.

Spring	Peak reaction force
Lateral	$F_{lateral} = \left[\frac{1}{0.228 \left(\frac{H}{D}\right) + 0.057} \right] [\gamma HDL \tan \varphi_{ps,p}]$ <p>Validity: up to $\begin{cases} H/D = 7 \text{ to } 8 & \text{for loose to medium sand} \\ H/D = 13 \text{ to } 15 & \text{for dense sand} \end{cases}$</p>
Upward	$F_{upward} = \left[\left(1 - \frac{\pi D}{8H}\right) \left(1.175 + 0.711 \frac{H}{D}\right) \right] [\gamma HDL \tan \varphi_{ps,p}]$ <p>Validity: up to $\begin{cases} H/D = 7 \text{ to } 8 & \text{for loose to medium sand} \\ H/D = 13 \text{ to } 15 & \text{for dense sand} \end{cases}$</p>
Downward	$F_{downward} = +N_q \gamma HD + N_\gamma 0.5 \gamma D^2$ <p>with:</p> $N_q = e^{\pi \tan \varphi^*} [\tan(45^\circ + \varphi^*/2)]^2 \text{ for loose and dense sand}$ $N_q = e^{3.8 \varphi^* \tan \varphi^*} [\tan(45^\circ + \varphi^*/2)]^2 \text{ for medium sand}$ $N_\gamma = e^{0.18 \varphi^* - 2.5}$ <p>where:</p> $\tan \varphi^* = \left(\frac{\cos \psi \cos \varphi_{ps,p}}{1 - \sin \psi \sin \varphi_{ps,p}} \right) \tan \varphi_{ps,p}$ <p>Notation:</p> <ul style="list-style-type: none"> γ : dry unit weight of sand H : embedment depth measured from the pipe springline D : pipe diameter L : pipe length $\varphi_{ps,p}$: peak plane strain friction angle, being correlated to the friction angle (φ_{ds}) measure from direct shear tests as $\tan \varphi_{ds} = \frac{\cos \psi \cos \varphi_{ps,p}}{1 - \sin \psi \sin \varphi_{ps,p}}$ <p>with $\psi = 1.25(\varphi_{ps,p} - \varphi_{crit})$ being the dilation angle, and φ_{crit} is the critical state friction angle of sand</p>
Spring	Peak displacement
Lateral	loose $0.112H/D + 0.139$ medium $0.085H/D + 0.087$ dense $0.035H/D + 0.026$
Upward	$0.01H$ to $0.02H$ for dense to loose sand $< 0.1D$
Downward	loose $0.1D$ medium $0.1D$ dense $0.2D$

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1189 **Table 6.** Preliminary selection criteria for pipe protection measures.

Category	Criterion
1. Design	1.1 Compatibility with fault mechanisms
	1.2 Compatibility with pipe–fault crossing geometry
	1.3 Compatibility with pipe cross-section geometry and steel grade
	1.4 Requirement for sophisticated analysis
	1.5 Requirement for experimental verification
	1.6 Compatibility with codes
2. Construction	2.1 Ease of on-site application
	2.2 Requirement for special installation equipment
	2.3 Special requirements for transportation to the construction site
3. Procurement	3.1 Availability in the market
	3.2 Production upon request
	3.3 High cost of purchase
	3.4 High cost of installation

1191 **Table 7.** Illustrative examples of applying the preliminary selection criteria for pipe
 1192 protection measures

Measure	Design criteria					
	1.1	1.2	1.3	1.4	1.5	1.6
Wall thickness increase	+	+	+	x	x	+
Pipe placement within culverts	x ⁺	+	N/A	x	x	+
Backfilling with pumice	+	+	+	x	x	+
Introduction of flexible joints	+	+	N/A	+	+	x
Route changing with high radius	+	+	+	+	x	x
Measure	Construction criteria					
	2.1	2.2	2.3			
Wall thickness increase	+	x	x			
Pipe placement within culverts	+	+	+			
Backfilling with pumice	+	x	+			
Introduction of flexible joints	+	x	x			
Route changing with high radius	x	x	N/A			
Measure	Procurement criteria					
	3.1	3.2	3.3	3.4		
Wall thickness increase	+	x	x	x		
Pipe placement within culverts	+	+	+	+		
Backfilling with pumice	+	N/A	x	x		
Introduction of flexible joints	+	+	+	x		
Route changing with high radius	N/A	N/A	+	+		

Abbreviations: +: yes / compliance / required, x: no / not required, N/A: not applicable

⁺ Compatible only with strike-slip fault mechanism

Table 8. Brief practical guidelines for the numerical modeling of protection measures.

Measure	Category	FE model type	Modeling
Steel grade upgrade	Pipe strengthening	Beam-type / Continuum	Modify steel material properties
Wall thickness increase		Beam-type / Continuum	Increase thickness of cross-section / Increase thickness of shell
Pipe wrapping		Continuum	Introduce a shell layer outside the pipe shell for the composite wrap, introduce appropriate contact between pipe and wrap
Trench backfilling with tire-derived aggregate	Soil friction reduction	Continuum	Model the entire soil block and the trench geometry, use material properties for the backfilling from experimental results
Use of geotextile-lined trenches		Continuum	Model the interfaces at trench walls
Trench backfilling with loose granular soil		Beam-type / Continuum	Modify soil properties (soil springs for the beam-type model and material law of 3D-solid elements for the continuum model)
Excavation of a wider trench		Continuum	Model trench geometry
Pipe placing within culverts		Beam-type / Continuum	Do not model soil (remove soil springs / 3D-solid elements) along
Replace soil backfill with EPS geofom blocks		Continuum	Model trench geometry and backfill materials with detail
Device applying external pressure		Complex measures	Continuum
Pipe route change		Beam-type	Model the entire pipe route
Introduction of flexible joints		Beam-type	Model joints with springs (Melissianos et al. 2016; Valsamis and Bouckovalas 2020)
Pipeline offset at the crossing		Beam-type	Model with detail the entire pipe route (Hart et al. 2004)
Use field bends		Beam-type and continuum	Model the entire pipe route and assess the integrity of bends with detailed modeling (Vazouras and Karamanos 2017)
Pipe with predefined buckling pattern		Continuum	Model with detail the pipe shell (Hasegawa et al. 2014)