Impact of Soil–Pipe Interaction and Ground Settlement on the Structural Reliability of Buried Water Supply Pipelines

Randhir Kumar, Himanshu Kumar Yadav, Dr. Raja Sonal Anand, Garima Yadav, Dr. Manish Kumar Mandal, Dr. Shailesh Kumar

Abstract: *Buried water supply pipelines are routinely exposed to differential ground settlement from consolidation, groundwater drawdown, backfill compaction and nearby excavation, which induces longitudinal bending rarely considered in design. This paper quantifies how soil–pipe interaction and the magnitude and shape of a ground settlement trough govern the structural reliability of a continuous welded mild steel water main (323.9 mm outside diameter, 6.35 mm wall). The pipeline is modelled as an Euler–Bernoulli beam on nonlinear, asymmetric Winkler springs whose bearing, uplift and axial resistances are derived for loose sand, dense sand, soft clay and stiff clay, and is subjected to a Gaussian settlement trough together with an internal pressure of 0.6 MPa. Deterministic analyses show that first yield of the pipe wall is reached at maximum settlements of 153–203 mm for a trough width parameter of 5 m, and that narrow troughs are far more damaging than wide ones. Monte Carlo fragility analysis yields lognormal fragility curves with median capacities of 170–228 mm, and first-order reliability analysis shows that the reliability index falls below a target of 3.0 once the mean settlement exceeds about 55–75 mm (for example, $\beta = 2.24$ in dense sand and 2.84 in soft clay at 80 mm). The settlement magnitude and trough width together account for more than 90% of the importance factors, whereas soil strength uncertainty, yield strength, wall thickness and pressure contribute less than 10%. The results provide settlement thresholds and design charts for reliability-based assessment of water mains.*

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Introduction

Water supply pipelines are long, buried and geographically distributed assets that traverse soils of widely varying stiffness and strength. Unlike above-ground structures, their loading is dominated not by applied forces but by imposed ground deformation, so that their performance depends on how the surrounding soil and the pipe share the deformation [1]. Physically based and statistical reviews of water main deterioration identify soil movement, differential settlement and loss of bedding support as major contributors to longitudinal bending and circumferential failures, alongside corrosion and operating pressure [2], [3]. Probabilistic studies have since shown that failure risk depends strongly on uncertain soil and loading parameters and must be treated in a reliability framework rather than through single safety factors [4], [5].

Differential ground settlement is a particularly common hazard for urban water mains. It arises from consolidation of soft alluvial deposits, groundwater abstraction, poorly compacted trench backfill, nearby deep excavations and tunnelling beneath existing pipelines. Field evidence from earthquakes and ground failures demonstrates that permanent ground deformation is the dominant cause of damage to continuous and jointed pipelines [6], [7]. In the alluvial plains of eastern India, trunk mains laid in soft clays and loose silty sands are often affected by such settlement.

Water main design usually checks hoop stress and ring deflection but treats longitudinal settlement effects only implicitly. Studies of pipelines affected by tunnelling and faulting show that the longitudinal response is controlled by the relative stiffness of pipe and soil and by the nonlinearity of the soil reaction [8]–[10]. Few studies have combined such soil–pipe interaction modelling with a formal reliability analysis for water supply pipes subjected to settlement. This study addresses that gap with three objectives: (i) to develop and validate a nonlinear soil–pipe interaction model for a buried water main crossing a settlement trough; (ii) to quantify the effect of soil type, settlement magnitude and trough width on pipe stresses; and (iii) to evaluate fragility curves, reliability indices and the relative importance of uncertain variables for reliability-based assessment.

Literature Review

2.1 Soil–Pipe Interaction Under Ground Movement

Early analyses of jointed water mains idealised the pipe as a beam on an elastic foundation and showed that joint rotation and loss of support control bending stresses [11]. For tunnelling-induced settlement, Winkler and elastic continuum solutions have been compared, and design charts relating pipe bending strain to relative pipe–soil stiffness have been proposed, with the settlement profile represented by a Gaussian trough [8], [9], [12]. Analytical and finite element methods developed for fault crossings extended these ideas to large deformations, combining bending with axial membrane strain and nonlinear soil springs [10], [13].

2.2 Soil Resistance Characterisation

The force–displacement relationships of the soil springs underpin all beam-on-spring analyses. Large-scale tests established lateral resistance and its dependence on burial depth and soil density [14], uplift mechanisms in sand were quantified through image-based deformation measurement [15], uplift response in granular soil was reassessed at full scale [16], and axial resistance was shown to depend strongly on soil dilation [17]. Partial saturation further increases lateral resistance [18], and bearing and uplift resistances differ by up to an order of magnitude.

Table 1. Summary of representative studies on soil–pipe interaction and pipeline reliability.

Study	Hazard / loading	Method	Key contribution	Limitation addressed here
Rajani et al. [11]	Loss of support	Beam on elastic foundation	Joint rotation in water mains	No reliability analysis
Vorster et al. [9]	Tunnelling settlement	Elastic continuum	Design charts, relative stiffness	Linear soil, deterministic
Karamitros et al. [10]	Fault displacement	Analytical beam	Bending + axial strain	Steel gas pipes, faults only
Vazouras et al. [13]	Fault displacement	3-D finite element	Local buckling, strain limits	High computational cost
Sadiq et al. [4]	Corrosion, pressure	Monte Carlo	Probabilistic failure of cast iron	No soil–pipe interaction
Lanzano et al. [7]	Earthquake	Empirical fragility	Fragility of gas pipelines	Not settlement-specific

2.3 Reliability of Buried Pipelines and Research Gap

Reliability methods for buried pipes have mostly targeted corrosion and pressure-related failures [4], [5] or seismic fragility [7]. Table 1 summarises representative studies. Geotechnical parameters exhibit large inherent variability [19], yet

settlement-induced longitudinal failure of water mains has seldom been examined probabilistically. The present study combines a validated nonlinear soil–pipe interaction model with Monte Carlo and first-order reliability methods for this purpose.

Methodology

The study follows the workflow in Figure 1.

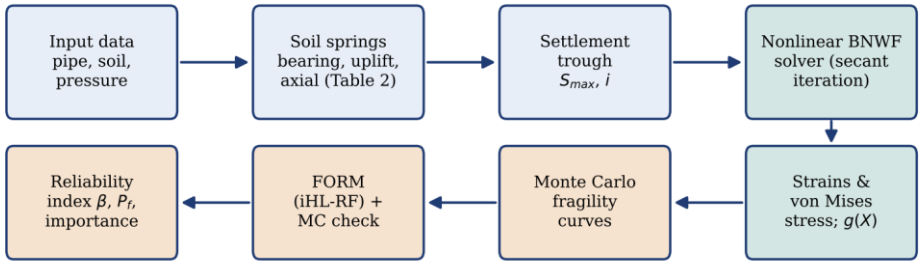


Figure 1. Methodology adopted for the reliability assessment of buried water pipelines under ground settlement.

3.1 Pipe and Settlement Model

The reference pipe is a continuous welded mild steel water main of outside diameter D = 323.9 mm and wall thickness t = 6.35 mm (E = 210 GPa, ν = 0.3, nominal yield strength 250 MPa) buried at a depth H = 1.5 m to the pipe centreline and operating at an internal pressure of 0.6 MPa, which produces a hoop stress of 15.0 MPa. The flexural rigidity is EI = 16.78 MN·m². The pipe crosses a settlement trough at right angles (Figure 2a), represented by the Gaussian profile widely used for ground movements [9], [12]:

$$s(x) = S_{\max} \exp(-x^2 / 2i^2) \quad (1)$$

where S_{max} is the maximum settlement and i is the distance from the trough centre to the point of inflection. The pipe deflection w(x) satisfies the beam-on-Winkler-foundation equation

$$EI\, d^4w/dx^4 + q(w - s) = 0 \quad (2)$$

in which q(·) is the nonlinear soil reaction per unit length. Equation (2) is solved with Hermite beam elements of 0.25 m length over an 80 m pipe segment, which is long enough for the pipe to be undisturbed at its ends. The nonlinear springs are handled by secant iteration until the change in deflection is below 10^{-7} m.

3.2 Soil Springs

The soil reaction is represented by elastic–perfectly plastic springs whose ultimate resistances and yield displacements follow formulations calibrated against the experimental studies reviewed in Section 2.2 and adopted in earlier pipeline analyses

[10], [14], [15]. Where the pipe moves down into the soil the vertical bearing spring acts; where the soil settles away from the pipe the overburden acts through the uplift spring, which is far softer and weaker (Figure 2b). The resulting asymmetry is central to the response. Axial friction t_u limits the membrane strain generated by geometric elongation of the deflected pipe, estimated as $\varepsilon_a = (\Delta L\ t_u / EA)^{1/2}$, where ΔL is the elongation obtained from the deflected shape. Table 2 lists the soil properties and the computed spring parameters.

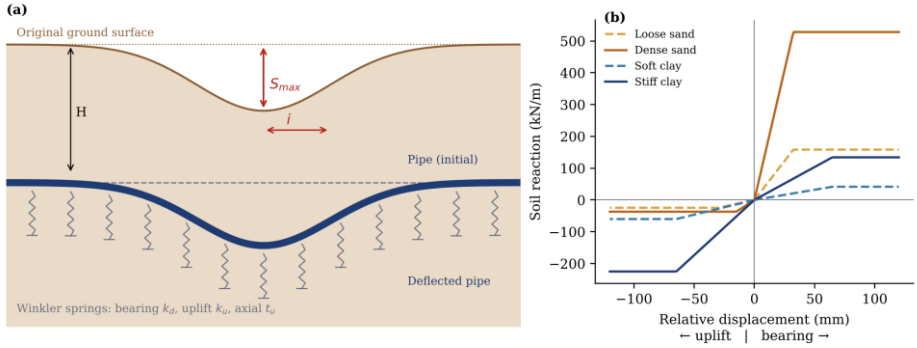


Figure 2. (a) Pipe crossing a Gaussian settlement trough, supported by soil springs; (b) bearing (+) and uplift (−) spring curves for the four soils.

Table 2. Soil properties and computed soil-spring parameters (D = 323.9 mm, H = 1.5 m).

Parameter	Loose sand	Dense sand	Soft clay	Stiff clay
Unit weight γ (kN/m³)	16	19	17	19
φ (°) / c_u (kPa)	30 / –	38 / –	– / 20	– / 75
Bearing capacity Q_d (kN/m)	158.3	528.2	41.6	134.1
Bearing yield disp. δ_d (mm)	32.4	32.4	64.8	64.8
Uplift capacity Q_u (kN/m)	24.5	36.9	60.0	225.0
Uplift yield disp. δ_u (mm)	30.0	15.0	64.8	64.8
Axial friction t_u (kN/m)	8.2	11.8	20.5	63.3

3.3 Limit State and Reliability Analysis

Longitudinal stress is the sum of axial membrane and bending components plus the Poisson effect of pressure, $\sigma_L = E(\varepsilon_a \pm \kappa D/2) + \nu\sigma_h$, where κ is the curvature. The limit state function is first yield of the pipe wall under the von Mises criterion:

$$g(X) = f_y - \max_x [\sigma_h^2 - \sigma_h\sigma_L + \sigma_L^2]^{1/2} \quad (3)$$

Six variables are treated as random (Table 3), with soil strength variability represented by a lognormal multiplier M_s on all spring capacities, consistent with reported coefficients of variation of geotechnical properties [19]. Fragility curves, $P_f = P[g < 0 \mid S_{max}]$, are computed by Monte Carlo simulation with 1,500 samples per settlement level and fitted with lognormal functions. Reliability indices for uncertain settlement are obtained with the first-order reliability method (FORM) using the Hasofer–Lind index [20] and the Rackwitz–Fiessler algorithm with a line-search modification [21], [22], checked against Monte Carlo simulation with 20,000 samples.

Table 3. Random variables used in the reliability analysis.

Variable	Symbol	Distribution	Mean	COV
Maximum settlement	S_{max}	Lognormal	50–120 mm	0.30
Trough width parameter	i	Lognormal	5.0 m	0.20
Soil strength multiplier	M_s	Lognormal	1.0	0.30
Yield strength	f_y	Lognormal	280 MPa	0.07
Wall thickness	t	Normal	6.35 mm	0.05
Internal pressure	p	Normal	0.6 MPa	0.10

Results and Discussion

4.1 Model Verification

With linear springs, Equation (2) has a closed-form solution in the Fourier domain for an infinite beam. Table 4 compares the numerical and analytical maximum deflections and bending moments. Deflections agree to within 0.001 mm and moments to within 0.15%, confirming the formulation and mesh.

Table 4. Verification of the numerical model against the analytical solution (linear springs).

Soil	S_{max} (mm) / i (m)	w_{max} model (mm)	w_{max} analytical (mm)	M_{max} model (kN·m)	M_{max} analytical (kN·m)	Error in M (%)
Loose sand	100 / 3	92.93	92.93	137.84	138.02	0.13
Dense sand	100 / 3	97.10	97.10	163.27	163.52	0.15
Soft clay	100 / 3	78.69	78.69	79.07	79.14	0.09
Stiff clay	100 / 3	88.02	88.02	113.82	113.95	0.11

4.2 Pipe Response to a Settlement Trough

Figure 3 shows the response to a trough with S_{max} = 100 mm and i = 5 m. The pipe follows the ground closely in dense sand (maximum deflection 97.5 mm) but lags

slightly behind it in soft clay (94.6 mm), because the softer and weaker soil allows relative movement. The bending moment is greatest at the trough centre (sagging) and reverses near ±9 m (hogging). Peak bending moments are 60.6 kN·m in dense sand and 52.1 kN·m in soft clay, giving peak von Mises stresses of 138.6 MPa and 125.8 MPa. Bending strain (585 με in dense sand) dominates the axial membrane strain (86 με), which is small because the elongation of a gently curved pipe is modest compared with the axial anchorage available.

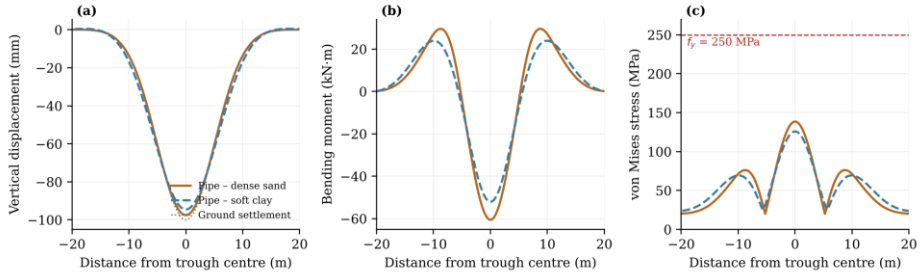


Figure 3. Response for S_{max} = 100 mm and i = 5 m: (a) ground settlement and pipe deflection; (b) bending moment; (c) von Mises stress.

Figure 4 plots the peak von Mises stress against settlement for the four soils. The stress increases almost linearly with S_{max}. First yield occurs at 153 mm in stiff clay, 179 mm in dense sand, 197 mm in soft clay and 203 mm in loose sand. Stiff clay is the most damaging because its high undrained strength gives strong uplift resistance, which forces the pipe to follow the trough. A linear-spring model for dense sand predicts yield at 167 mm, which is about 7% conservative because it ignores yielding of the soil.

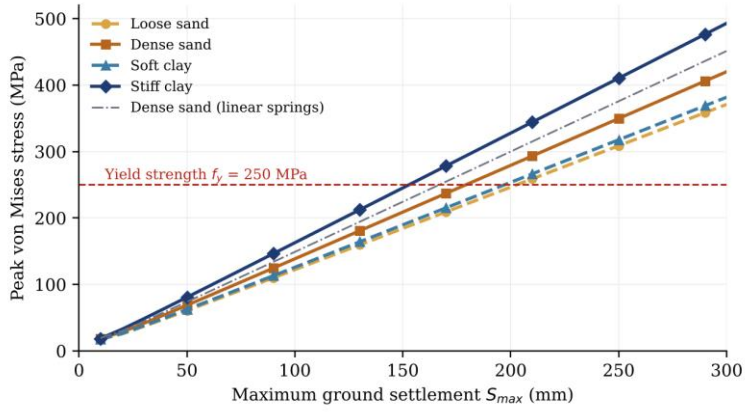


Figure 4. Peak von Mises stress versus maximum ground settlement (i = 5 m).

The shape of the trough is as important as its depth. Figure 5 maps the stress ratio $\sigma_{vm}/\bar{f}_y$ over S_{max} and i. Because bending curvature scales approximately with S_{max}/i^2,

narrow troughs ($i \leq 3$ m), typical of trench-backfill subsidence and local excavation, cause yield at only 80–90 mm of settlement in dense sand and about 125 mm in soft clay, whereas troughs wider than about 8 m remain below half the yield strength for settlements up to 250 mm.

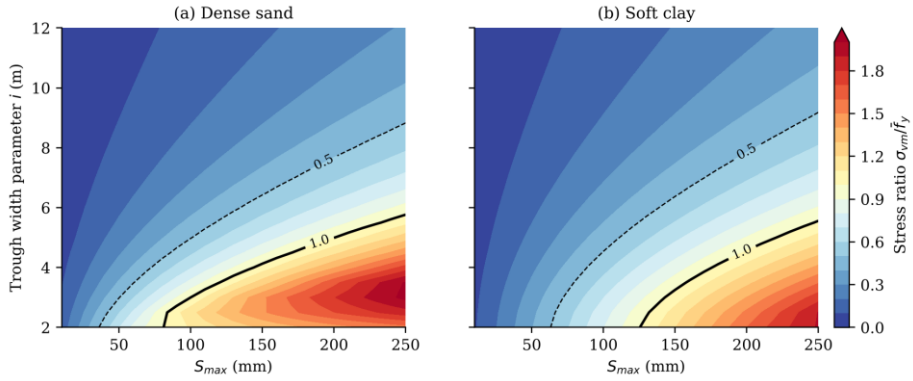


Figure 5. Design chart of stress ratio as a function of maximum settlement and trough width: (a) dense sand; (b) soft clay. The bold contour marks first yield.

4.3 Fragility Analysis

Figure 6 presents the fragility curves, which give the probability of first yield conditional on S_{max} while the other five variables vary randomly. The lognormal fits have median capacities θ of 170 mm (stiff clay), 197 mm (dense sand), 224 mm (soft clay) and 228 mm (loose sand), with dispersions ζ of 0.24–0.31. The probability of yield is negligible below about 80 mm in all soils but rises steeply beyond 120 mm, so a settlement of about 100 mm can be regarded as a practical alert level for the reference pipe.

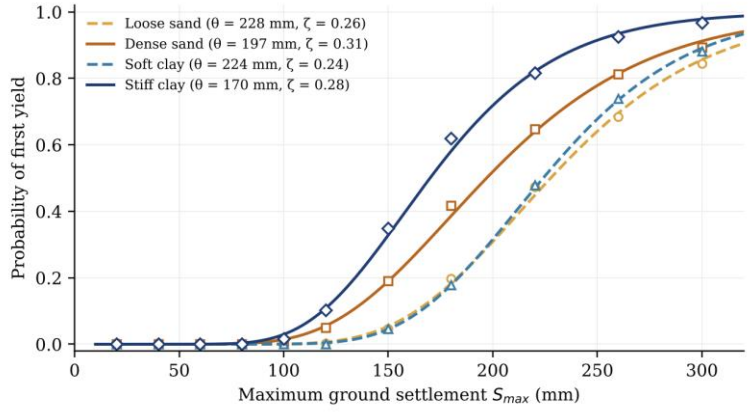


Figure 6. Fragility curves for first yield of the pipe wall (markers: Monte Carlo; lines: lognormal fits).

4.4 Reliability Index and Sensitivity

When the settlement itself is uncertain (lognormal, COV = 0.30), FORM gives the reliability indices in Table 5 and Figure 7a. For a mean settlement of 50 mm, all soils except stiff clay exceed a target index of $\beta_T = 3.0$, a value commonly adopted for ultimate limit states [22]. At 80 mm the index drops to 1.97–2.84, and at 120 mm to 0.94–1.72. FORM is in close agreement with Monte Carlo simulation: for dense sand at 80 mm the simulated failure probability is 9.25×10^{-3} against 1.27×10^{-2} from FORM, and for soft clay at 120 mm 4.14×10^{-2} against 4.32×10^{-2}.

Table 5. Reliability index β and probability of failure P_f (in brackets) for different mean settlements.

Soil	$\bar{S}_{max}$ = 50 mm	$\bar{S}_{max}$ = 80 mm	$\bar{S}_{max}$ = 120 mm
Loose sand	4.05 (2.53×10^{-5})	2.77 (2.81×10^{-3})	1.69 (4.56×10^{-2})
Dense sand	3.42 (3.17×10^{-4})	2.24 (1.27×10^{-2})	1.25 (1.06×10^{-1})
Soft clay	4.16 (1.59×10^{-5})	2.84 (2.28×10^{-3})	1.72 (4.32×10^{-2})
Stiff clay	3.20 (6.94×10^{-4})	1.97 (2.44×10^{-2})	0.94 (1.74×10^{-1})

The importance factors in Figure 7b show that the settlement magnitude ($\alpha^2 = 0.52$–0.67) and trough width (0.23–0.43) together account for more than 90% of the uncertainty. Soil strength contributes at most 0.06, yield strength about 0.03, and wall thickness and pressure are negligible. Reliability therefore depends far more on how well the settlement field is predicted or monitored than on refining the pipe material or soil spring parameters, which reflects the distinction between reducible and inherent uncertainty in reliability assessment [23].

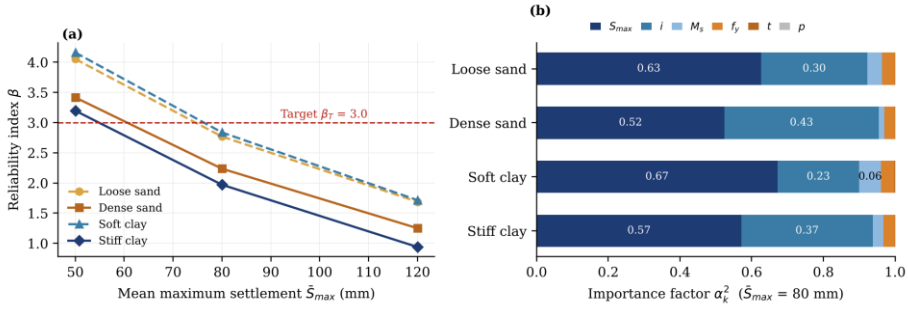


Figure 7. (a) Reliability index versus mean maximum settlement; (b) FORM importance factors for a mean settlement of 80 mm.

4.5 Effect of Pipe Diameter and Burial Depth

Table 6 shows the influence of pipe size and depth for a mean settlement of 80 mm. In dense sand, reliability falls as diameter increases (β = 3.09 to 2.00) because the stiff soil forces even a large pipe to follow the trough while the outer-fibre strain grows with D. In soft clay the trend reverses for the largest pipe (β = 3.60): the weak soil yields before the pipe does, so the pipe bridges the trough, as predicted by

relative-stiffness solutions [8], [9]. Deeper burial increases the uplift resistance and lowers β in both soils, by 0.47 in dense sand and 0.27 in soft clay between 1.0 m and 3.0 m depth.

Table 6. Effect of pipe diameter and burial depth on the reliability index ($\bar{S}_{max}$ = 80 mm).

Parameter	Value	β – dense sand	β – soft clay
Diameter (D × t)	168.3 × 4.50 mm	3.09	3.17
	323.9 × 6.35 mm	2.24	2.84
	610.0 × 7.92 mm	2.00	3.60
Burial depth H	1.0 m	2.44	3.03
	1.5 m	2.24	2.84
	2.0 m	2.12	2.79
	3.0 m	1.97	2.77

Practical Implications

The results lead to several recommendations for water utilities and designers. First, settlement troughs should be assessed by both depth and width: the stress ratio chart (Figure 5) and fragility curves (Figure 6) can be used for screening, with troughs narrower than about 3 m and deeper than about 80 mm warranting detailed analysis. Second, because S_{max} and i dominate the importance factors, investment in settlement prediction and monitoring near excavations, dewatering and tunnelling yields greater reliability gains than increasing wall thickness. Third, uniform backfill and gradual transitions between support conditions are preferable to abrupt changes in soil stiffness. Finally, flexible joints should be considered for large pipes in stiff ground.

Limitations and Future Work

The analysis is elastic and uses first yield as the limit state, which is conservative for ductile steel pipes able to sustain strains well beyond yield. The pipe is assumed continuous and welded; jointed ductile iron and plastic pipes require joint rotation and pull-out limit states [11]. Spatial variability of soil along the pipe and oblique troughs are not modelled. Future work should add elastoplastic strain-based limit states, random-field soil properties, corrosion-induced wall loss and validation against instrumented field pipelines.

Conclusion

A validated nonlinear soil–pipe interaction model combined with Monte Carlo and FORM analysis leads to the following conclusions. (1) Pipe stress increases almost linearly with settlement, and first yield of a 323.9 mm main occurs at 153–203 mm

of settlement for a trough width of 5 m, with stiff clay the most onerous soil because of its high uplift resistance. (2) Narrow troughs are critical; bending demand scales with S_{max}/i^2, so the trough width must be considered alongside its depth. (3) Lognormal fragility curves give median settlement capacities of 170–228 mm, and the reliability index falls below 3.0 once the mean settlement exceeds about 55–75 mm. (4) Settlement magnitude and trough width account for more than 90% of the importance factors, so settlement prediction and monitoring are the most effective means of improving reliability. (5) Soil–pipe relative stiffness governs the effects of diameter and depth, so that large pipes and deep burial are most vulnerable in stiff soils.

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