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FAILURE MECHANISMS OF PIPELINES SUBJECTED TO LOSS OF SUPPORT UNDER DEEP BURIAL CONDITIONS

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ABSTRACT

This paper investigates the failure mechanisms of pipelines subjected to a localized loss of support. An experimental program was conducted, which consisted of a series of four centrifuge model tests containing an aluminum tube embedded in a pure dry sand backfill that was placed over an underlying rectangular rigid base moving downwards during the test. All models were built taking advantage of the longitudinal symmetry of the problem. The prototype pipe had a diameter (D) of 1.1 m and a soil cover height of about 5 D, characterizing deep burial conditions. Failure patterns were observed within a vertical section comprising the central axis of the pipe and also in four distinct vertical transverse sections along the length of the pipe in the region of ground loss. The influence of pipe stiffness and backfill density on the behavior of the system was assessed. The transverse sections showed fully developed slip surfaces starting in the vicinity of the edge of the void towards the adjacent soil mass. The mode of failure of the flexible pipes took the form of a severe deformation at the region of the shoulder and a reversal of curvature at the invert due to over-deflection. This situation was more critical in the central section. The damage experienced by the flexible pipes was noticeably more pronounced when using the looser backfill, whereas only negligible deflections were observed when using the denser backfill. The experimental results were compared with analytical predictions, which showed to be highly unconservative for the case loose backfill.

INTRODUCTION

The crowded infrastructure of the large urban areas around the world is requiring the installation of underground pipelines in increasingly higher depths. In conditions where the soil envelope is homogeneous and properly compacted, it has been observed that the stress distribution in the vicinity of the pipe becomes more homogeneous for higher depths of soil cover [1]. Katona [2] points out that the magnitude of the thrust stress is the governing criterion in the design of corrugated polyethylene pipes with cover depths larger than 12 m.

However, due to their long length, buried pipelines have high probability of experiencing localized losses of support during their lifetime. Mining activities, collapsible soils, karstic soils, and voids due to piping are the most usual causes of loss of support of underground pipelines. The overall scenario after ground loss is characterized by a complex three-dimensional soil-structure interaction, which includes redistribution of the stresses in the pipe wall and in the surrounding soil. The pipe may undergo high circumferential and longitudinal bending moments in the region of the void, which can eventually cause failure of the structure. If the pipe is located at significant depths, the situation may be aggravated due to the high confinement.

This paper presents an experimental study on the failure mechanisms of pipes subjected to loss of support under deep burial conditions. The experimental program consisted of a series of four centrifuge model tests containing an aluminum tube buried in a pure dry sand backfill, and resting over a rigid base that moved downwards during the test. Failure patterns were observed within a vertical section comprising the central axis of the pipe and also in four distinct transverse vertical sections along the axis of the tube. The influence of pipe stiffness and backfill density on the behavior of the soil-pipe system is evaluated.

NOMENCLATURE

Δ_c = vertical displacement of pipe crown

Δ_i = vertical displacement of pipe invert

$\Delta = \Delta_c + \Delta_i$

Δ/D = vertical deflection of pipe cross-section

δ = displacement of the rigid base

θ = angle of inclination to the horizontal of the outermost slip surface near the edge of the void

ϕ' = soil angle of internal friction

δ_s = settlement at free surface of backfill

δ_p = settlement of the neutral surface of the pipeline

ν_s = Poisson ratio of backfill soil

D = pipe diameter

D_r = backfill relative density

E = Young's modulus of pipe

EI = pipe flexural stiffness

E_s = Young's modulus of backfill soil

H = height of soil cover above pipe crown

I = centroidal moment of inertia of pipe cross-section

M = bending moment

h = height of the failure zone in the soil mass

k = modulus of subgrade reaction

L = pipe length

L_v = length of the void

p = reaction force

q = uniformly distributed load

t = pipe wall thickness

c_x = length along x-axis of the projection of the failure zone in x-z plane, measured from the edge of the void

c_y = length along y-axis of the projection of the failure zone in x-z plane, measured from the edge of the void

CENTRIFUGE MODEL TESTS

The centrifuge models were built taking advantage of the longitudinal symmetry of the problem. The models were constructed in an aluminum container with inside dimensions of 419 x 203 mm in plane and 300 mm in height. The front wall of the container consisted of a transparent Plaxiglass plate to enable in-flight visualization of the models during the tests. The aluminum plates were lined with a PTFE layer to minimize side wall friction, whereas two films of polyester were used in the Plexiglass. The bottom of the container had a rectangular slot where an aluminum rigid base with dimensions of 85 x 17.5 mm was able to slide vertically. The moving base was located adjacent to the transparent wall, and its lowering was controlled by an electromagnet. The geometric configuration of the models with dimensions at the prototype scale in meters is shown in Fig. 1.

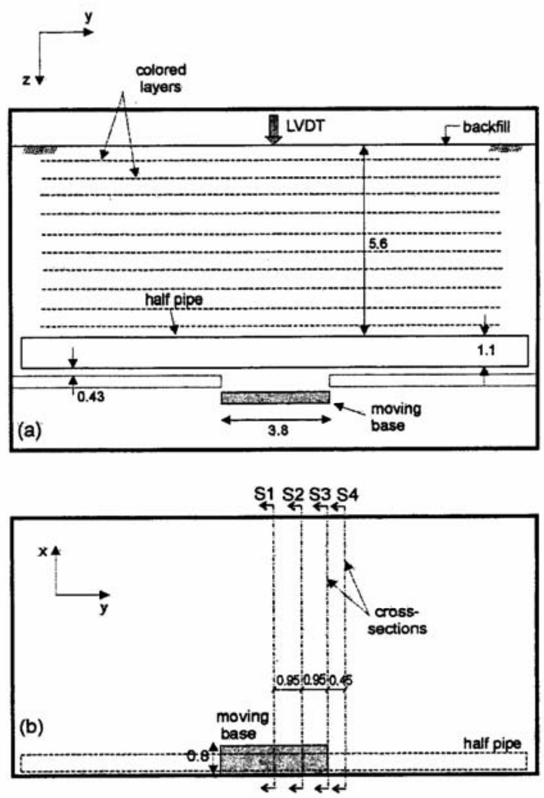

Figure 1. a) Scheme of model configuration; b) plan view of model showing positions of the transverse sections. Prototype dimensions (m)

The model pipes used in the tests consisted of half-sections of aluminum tubes with smooth surface. The pipes had the following dimensions: 25.4 mm in outside diameter (D), 400 mm in length (L), and 0.127 or 0.7 mm in wall thickness (t). These thickness values were selected to represent flexible (F) and rigid (R) pipes, respectively. The model pipes were displaced against the transparent wall, over the moving base, resting on a bedding of sand with constant thickness of 9.5 mm.

Table 1 shows the characteristics of the pipes. The subscripts m and p denote model and prototype, respectively, and EI represents the flexural stiffness per unit length. Although the edges of the half section of the pipe were not prevented from rotating during the test, the measured rotation after failure was negligible (less than 1^0.)

Table 1. Characteristics of the model pipes

Pipe type	D_m (mm)	D_p (m)	t_m (mm)	t_p (mm)	EI_m (N·m)	EI_p (kN·m)
F	25.4	1.14	0.127	5.7	$1.23 \cdot 10^{-2}$	1.12
R	25.4	1.14	0.7	31.5	2.06	187.5

The centrifuge tests were performed in the 15 g-ton centrifuge of the University of Colorado at Boulder. This equipment is a Genisco 1230 rotary accelerator with a nominal radius of 1.36 m, and with capability of accelerating a 135-kg payload to 100 g. All models were tested under a nominal centrifuge acceleration of 45 g.

In order to achieve homogeneous densities, the models were carefully prepared by pluviating previously air-dried sand into the assembled testing apparatus. Selected relative densities (D_r) of 85 and 42% were achieved by raining the sand from calibrated constant heights and with calibrated constant discharge rates. A final prototype height of soil cover (H) of 5.6 m was ensured for all models, corresponding to a cover depth ratio H/D of approximately 5. Installations are typically considered in deep burial conditions when the geometric ratio H/D reaches values of approximately 2.5 [3, 4, 5].

During the tests, the power of the electromagnet was turned off causing the plunge of the moving base and the creation of a void in the soil mass below the pipe. A fixed falling height of 20 mm was ensured for all models, which corresponded to 0.9 m in prototype dimension. The development of the failure mechanisms was inferred from the pattern of ten thin horizontal layers of colored sand placed within the soil at vertical intervals of 12.7 mm (Fig. 1a). The displacements of the colored layers were photographically recorded in the y-z plane through the transparent wall of the strongbox. Pictures of the transparent wall were taken before and after lowering the base at the target centrifugal acceleration.

The final procedure consisted of carefully wetting and then dissecting the model in order to investigate the patterns within the sand mass along the y-axis. Displacements of the colored layers in this direction were photographically recorded in the four different sections (S1, S2, S3, S4) identified in Fig. 1b. The first three sections were located at the center of the void (S1), at one of the edges (S3) and at mid-way between both positions (S2). The fourth section (S4) was excavated in the outer soil mass.

Settlement at the free surface of the backfill (δ_s) was recorded with one LVDT mounted on the top of the strongbox. The rod of the LVDT was positioned at the center of box, 9 mm from the acrylic wall.

All models were prepared using dry Ottawa F-75 sand. The material is a fine uniformly graded quartz (silica) sand with an average particle size of 0.22 mm, grain specific gravity of 2.65 and maximum and minimum void ratios of 0.805 and 0.486, respectively. The grain size distribution of the sand is shown in Fig.2. The unit weight values of the material corresponding to the target relative densities of 42 and 85% are 15.86 and 17.28 kN/m^3, respectively. The angle of internal friction (ϕ) of the sand, obtained from compression plane strain tests [6], equals 40 and 48^0 for D_r = 42 and 85%, respectively. The Young's modulus (E_s) corresponding to 50% of the maximum deviatoric stress equals 26.8 and 44.4 MPa, for D_r = 42 and 85%, respectively. Further information about the characteristics of the testing program can be found in Costa [7].

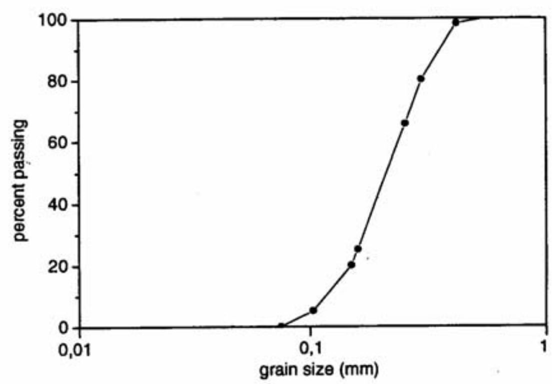

Figure 2. Grain size distribution of the sand used as backfill

A total of four model tests were performed in this experimental investigation. The characteristics of the tests are summarized in Table 2. A relative density of 42% was selected to represent a loose backfill, while a relative density of 85% was chosen to represent a dense backfill.

Table 2. Test characteristics

Test	D_r (%)	Pipe type	g-level	δ (m)
1	42	F	45	0.9
2	85	F	45	0.9
3	42	R	45	0.9
4	85	R	45	0.9

FAILURE MECHANISMS IN THE SOIL MASS

Figures 3 to 6 show the failure patterns observed in sections S1, S2 and S3 of the models. The slip surfaces in the soil mass are represented by dashed lines, and the zone of failure is shaded. Patterns composed by three distinct slip surfaces propagating from the region near the edge of the void are observed in all investigated situations. The innermost slip surface is formed by the flow of the material adjacent to the pipe towards the underlying void. The mechanism of formation of the other two slip surfaces is basically related to the instability of the soil mass next to the moving base, which is activated by the continuous migration of the material over the base to the created void and by the deformations of the wall of the pipe.

The geometric characteristics of the failure zones in the backfill soil and the settlement measured at the surface of the backfill (δ_s) (as a percentage of the underlying base displacement δ_r) are given in Table 3. The nomenclature for the parameters is indicated in Fig. 3, section S1.

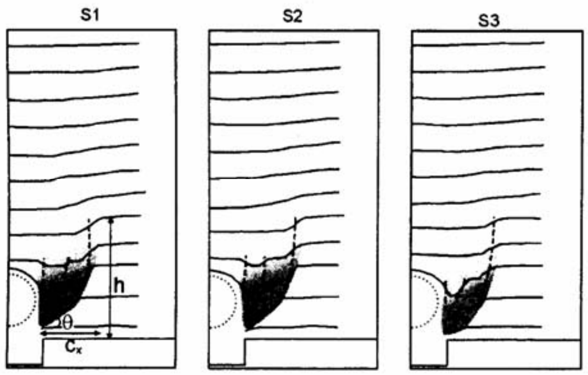

Figure 3. Failure mechanisms of the transverse sections of Model 1 (flexible pipe in loose backfill)

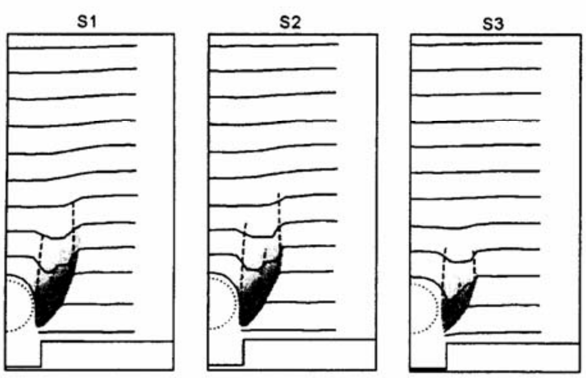

Figure 4. Failure mechanisms of the transverse sections of Model 2 (flexible pipe in dense backfill)

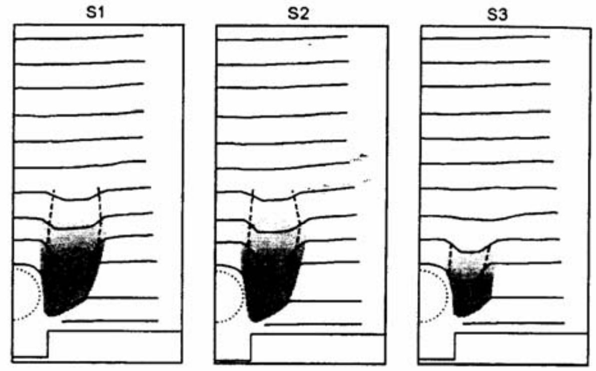

Figure 5. Failure mechanisms of the transverse sections of Model 3 (rigid pipe in loose backfill)

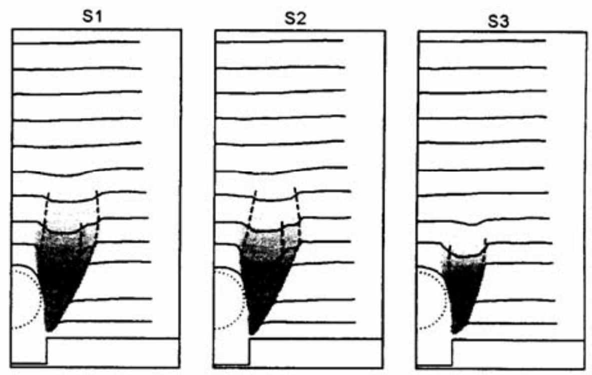

Figure 6. Failure mechanisms of the transverse sections of Model 4 (rigid pipe in dense backfill)

The following observations can be drawn from the results shown in Figures 3 to 6 and Table 3 regarding the influence of pipe stiffness and soil density. A localized loss of support experienced by the flexible pipe in the loose backfill caused a failure zone with a smaller height (h) and horizontal extent (c_x) in comparison with the observed for the rigid pipe embedded in the loose backfill. Likewise, the response of the flexible pipe undergoing loss of support in the dense backfill is characterized by a failure zone with smaller c_x and approximately equal h with respect to the failure zone of the rigid pipe in the dense backfill.

The influence of pipe stiffness on surficial settlements is negligible, once δ_s measured at the dense and the loose backfills was either equal or very close, irrespective to the stiffness of the embedded pipe (Table 3).

Table 3. Geometric characteristics of the failure zones in x-z plane

Test	D_r (%)	Pipe type	θ (^{0})	x_i/D	h/D	δ_s/δ (%)
1	42	F	38	1.6	4.4	18.0
2	85	F	54	1.1	5.0	5.5
3	42	R	50	1.9	5.1	16.0
4	85	R	56	1.7	4.9	5.5

No clear patterns of slip surfaces were verified in the longitudinal section of the models (plane y-z), but only zones of subsidence in the soil mass due to the migration of the material to the underlying void. Figures 7 and 8 show the recorded longitudinal failure patterns in Model 1 and 2, respectively. In the same way as in the transverse direction, the extension of the failure zone was quantified by the length of its projection in the x-y plane (c_y), measured from the edge of the void. In the loose backfill c_y is about 0.5D and 0.25D with the flexible and the rigid pipe, respectively. Conversely, c_y is negligible in the dense backfill, regardless the stiffness of the pipe. A comparison of Figures 7 and 8 shows that the zone of subsidence within the soil is more pronounced in the loose backfill, which is in agreement with the surficial settlement values given in Table 3.

The failure patterns in all model tests also included the development of small cavities near the edges of the void, as indicated in Figures 7 and 8.

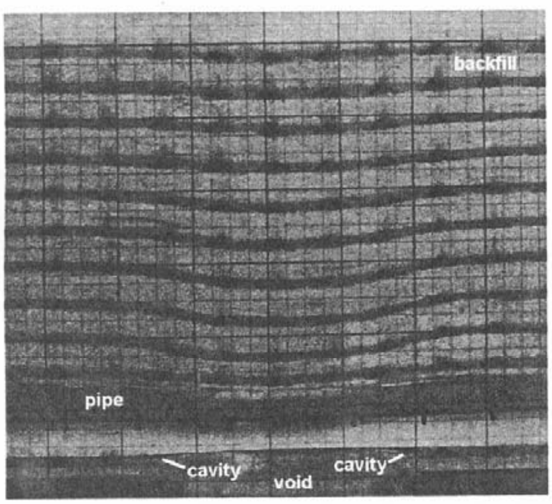

Figure 7. Failure pattern in longitudinal section of model 1 (flexible pipe in loose backfill)

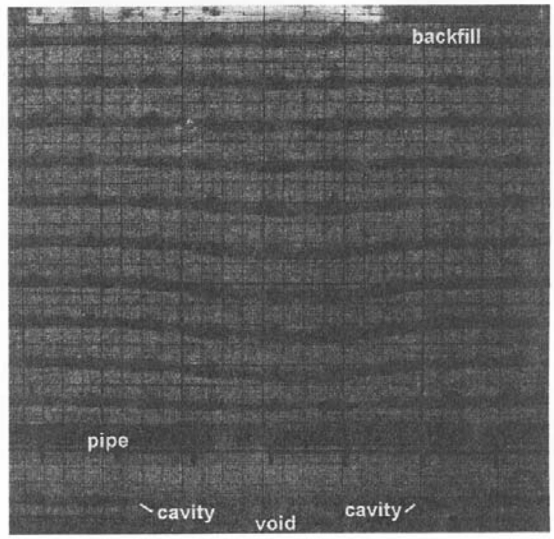

Figure 8. Failure pattern of longitudinal section of model 2 (flexible pipe in dense backfill)

EDGE EFFECTS

Sections S1 and S2 showed similar patterns of slip surfaces in Figures 3 to 6, suggesting that plane strain conditions can be assumed, at least, within the middle third of the length of the void. On the other hand, very different patterns of shear bands were observed in the soil mass immediately above the edge of the void (section S3). For a fixed base displacement (δ), S3 showed less developed slip surfaces when compared to the sections closer to the center of the void. The height of the zone of failure (h) at the edge is 15 to 30 % smaller than that observed in sections S1 and S2. Model 1 is the only exception, which showed a failure pattern in S3 comparable to that of S1 and S2.

The propagation of less developed slip surfaces at the edges of the structure is a direct consequence of the friction between the collapsing soil mass above the void and the stable exterior material in the imminence of collapse. Friction between the two masses restricts the movements of the soil particles that tend to follow the downward translation of the base.

No discontinuities were verified in section S4 in the outer soil mass in any analyzed condition.

FAILURE OF THE FLEXIBLE PIPES

The mode of failure of the flexible pipes embedded in the loose backfill (model 1) took the form of buckling at the region of the shoulder and the crown, and a reversal of curvature at the invert due to over-deflection. This scenario was more critical at the central section S1, but gradually attenuated towards the edges of the void. However, the largest damage was observed in the shoulder of the pipe. Figure 9 shows a view of the pipe after the test, recorded from one of its edges. The extension of the damage of the shoulder at the region of section S1 is indicated with arrows. At S1, the vertical deflection (Δ/D) of the pipe caused by the movement of the underlying base was approximately 26%.

Despite its low stiffness, the flexible pipe embedded in the dense backfill suffered virtually no deformations, as shown in Fig. 8.

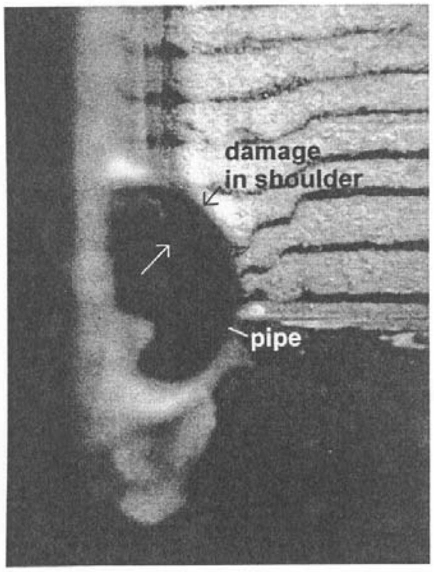

Figure 9. Damage experienced by the flexible pipe in the loose backfill (Section 1)

PREDICTION OF SETTLEMENTS OF FLEXIBLE PIPELINES

Buried pipeline research has gained substantial impulse since the 1920s with the pioneer work of Marston and his associates about external loads on rigid pipes. Research on flexible pipes was initiated about two decades later by Spangler [8], which devised a method to predict the deflections of flexible pipes based on extensive field and laboratory observations. New efforts on the study of the behavior of flexible pipes took place in the late 1950s [9]. This period coincided with the development of new theories on the failure by buckling [10, 11, 12] and hoop compression [13].

Despite the developments achieved throughout the last century on several aspects of the buried pipe field, very little attention has been paid to the longitudinal behavior of such structures [14]. Although equally important as the ring behavior, the longitudinal effects are seldom taken into account in the geotechnical design of pipes and culverts [15]. Usually, this topic is either briefly mentioned or completely overlooked by the current standards and codes of practice.

A few analytical methods to predict settlement of buried pipelines are available in literature. In a simplistic approach, a group of methods assume the pipeline as a beam resting on fixed or hinge supports [16, 17, 18]. The settlement is thus calculated with the equation of the elastic line:

$$\frac{d^2 z}{d\delta_p^{\ 2}} = \frac{M}{EI} \tag{1}$$

However, buried pipes are in direct contact with the supporting medium, so that continuously distributed reaction forces are produced upon deflection. Another group of solutions [19, 20, 21, 22] take this aspect into consideration assuming that the intensity of the reaction force (p) at any point is proportional to the settlement (δ_p) of the pipe at this point. That is:

$$p = k \cdot \delta_p \tag{2}$$

Equation (2) is known as the Winkler's Hypothesis, and k is the modulus of subgrade reaction, which incorporates the geometric characteristics of the structure.

Particularly, combining the elastic line equation and the Winkler's hypothesis, the settlement of a buried pipeline with a uniformly distributed load q and crossing a void with length L_v can be calculated using expression (3) [19]. The settlement of the pipe at the center of the void can then be determined knowing the bending moment and the shear force in the region of the pipe over the edges of the void.

$$\delta_p = \frac{q\lambda L_v}{k}\left[e^{-\lambda y}\cos\lambda y - \alpha\left(e^{-\lambda y}(\cos\lambda y - \sin\lambda y)\right)\right] \tag{3}$$

where:

$$\lambda = \sqrt[4]{\frac{k}{4EI}} \tag{4}$$

$$\alpha = \frac{6 - \lambda^2 L_v^2}{6(2 + \lambda L_v)} \tag{5}$$

The modulus of subgrade reaction can be estimated as [23]:

$$k = 0.65\sqrt[12]{\frac{E_s D^4}{EI}}\frac{E_s}{1 - \nu_s^2} \tag{6}$$

A prediction of the settlement of the flexible pipe at section S1 in models 1 and 2, along with comparisons with the observed experimental results is given in Table 4. The values assigned for the required parameters for the calculations are also listed in Table 4. Δ_c and Δ_i stand for the displacements of pipe crown and invert, respectively, measured in the tests. The analytical method provides highly unconservative results when the pipe is embedded in loose backfill (model 1) and results on the safe side with dense backfill (model 2). These comparisons clearly points out the main shortcoming of the available analytical methods: that the computed settlement represents the displacements of the neutral surface of the pipeline only. The ring deflections are completely neglected, which leads to inconsistent results. As shown in Fig. 7, although virtually no vertical displacement is observed in the longitudinal axis, the pipe embedded in the loose backfill experiences severe deformation in the region over the void.

Table 4. Predicted and observed settlements in the pipe

Test	E_s (MPa)	ν_s	k (MPa)	q (kN/m)	δ_p (mm)	Δ_c (mm)	Δ_i (mm)
1	26.8	0.36	17.4	100	6	234	69
2	44.4	0.26	28.1	110	4	-	-

CONCLUSIONS

A series of four centrifuge tests was conducted to investigate the behavior of pipelines subjected to ground loss of support under deep burial conditions. The failure mechanisms that took place due to the downward movement of an underlying rigid base beneath the pipe were investigated in longitudinal and transverse sections of the models.

Analysis of the results showed that the failure patterns in the soil mass is characterized by the formation of three distinct slip surfaces propagating from the region near the edge of the void. The zone of failure extended to the external soil mass adjacent to the void. The presence of rigid pipes caused comparatively larger zones of failure in both loose and dense backfills. The magnitude of the surficial settlement showed to be independent of the pipe stiffness.

The flexible pipes embedded in the loose backfill experienced some damage at the region of the shoulder and the crown, and a reversal of curvature at the invert due to over-deflection. The largest damage was observed in the region of the shoulder. The flexible pipe embedded in the dense backfill suffered negligible deflections due to the loss of support.

Analytical solutions for calculating settlements of buried pipelines can provide inconsistent results, once ring deflections are not taken into consideration in the methods. For the cases investigated herein, the used analytical method furnished highly unconservative results with loose backfill and results on the safe side with dense backfill.

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