

Buried pipelines subjected to landslide-induced actions

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ABSTRACT

An original Finite Element methodology is proposed to rigorously simulate the response of a buried pipeline subjected to landslide-induced actions. The problem is decoupled and a two-step procedure is adopted that allows the correct simulation of both the soil sliding process and of all possible modes of pipeline failure. In the first step, a "global" free field model is employed to simulate the evolution of soil movement during the landslide event. In the second step, an extremely fine "local" model is introduced comprising the pipeline and portion of the surrounding soil. The computed displacement time histories from the first step (global model) are imposed as boundary conditions to the local model. The methodology is utilized to investigate the structural performance of a buried steel pipeline subjected to a rotationally evolving landslide. It is concluded that, during the landslide event, the pipeline is subjected to a pull-out action that tends to drag the pipeline downslope, inducing tension near the landslide crown and compression near the landslide toe and a lateral push at the area of the landslide toe that induces severe bending on the pipe. The combination of bending and compression at the area of the landslide toe provokes buckling of the curved fitting component.

Keywords: pipe, 3D finite element modeling, buckling

INTRODUCTION

Geohazards, which are frequently documented as the main cause of pipelines failures [e.g., Hamada and O'Rourke (1992); O'Rourke and Liu (1999)], are defined as large ground displacements arising from: slope failure, creep, earthquake triggered slope movement, liquefaction-induced lateral spreading, surface faulting and subsidence. Among the various types of permanent ground displacement, the effects of landslide occurrence on a pipeline are the ones attracting the designer's interest. This is not only due to the higher frequency of occurrence compared to other types of permanent ground (fault rupture, lateral spreading), but also because typically pipelines are very long (e.g. hundreds of miles for hydrocarbon transmission pipelines) and hence they will unavoidably cross several precarious slopes along their route.

Determining the behavior of pipelines subjected to landsliding is a great challenge and remains a matter requiring further research. The current state of practice addresses the pipeline-landslide interaction in a rather simplifying manner. As with any Permanent Ground Deformation (PGD) problem, a Winkler-type beam-on-spring methodology is assumed. The soil is often idealized by non-linear discrete soil springs (acting in the axial, lateral, upward, downward direction), while the landslide movement is represented as a set of displacements applied to the free- nodes of the respective springs [Jibson & Keefer (1993); Gantes et al. (2008)]. Notwithstanding the practical value of Winkler-type methodologies, the latter are as accurate as the assumed p-y springs formulations, while they completely ignore any load transfer effects between the adjacent soil springs. For design purposes, the elastic perfectly plastic (p-y) formulations suggested by ALA (2001) and PRCI (2004) are commonly adopted, although recent research findings are questioning their appropriateness. Phillips et al. (2004), Yimsiri et al. (2004), Guo (2005), Hsu et al. (2006), Cocchetti et al. (2009), Daiyan et al. (2009, 2010) and Pike & Kenny (2011) systematically estimated (both numerically and experimentally) the oblique pipeline-soil capacity (i.e. under axial-lateral and lateral-vertical displacement)

and concluded that the bearing mechanisms under combined loading are strongly coupled; as such it may be inaccurate to replace soil by independent uniaxial p-y springs.

Scope of this the paper is to: (i) propose an original Finite element methodology that allows the study of the coupled soil-pipeline problem, (ii) enhance our understanding on the mechanics governing the soil-landslide interplay, and (iii) provide evidence on the importance of the detailing (i.e. location and geometry of fitting components (elbows and bends)) on the actual capacity of the pipeline-soil system.

PROBLEM STATEMENT

This study numerically investigates the structural performance of a buried steel pipeline subjected to a rotationally evolving landslide (Fig. 1). For the purposes of this study a rather typical hydrocarbon transportation pipeline is considered. The pipeline is assumed to be made of steel of grade X65 ($\sigma y = 450$ MPa). It has been designed for maximum operating pressure pmax = 9 MPa. Assuming an outer diameter of D = 36 in, its thickness is t = 12.7 mm as dictated by the expression:

$$p_{max} = 0.72^* (2\sigma_y t/D) \tag{1}$$

To render the results more realistic the pipeline is considered to function under operating pressure poper = 6 MPa. It is embedded within the soil at depth of H = 1.5 m from the surface to the pipe crown. Along its route, the pipeline crosses a slope that is considered prone to failure. The upper soil layer, termed the "unstable" soil, overlies the "stable" soil layer. Sandwiched between these two layers, lies the potential sliding surface of reduced strength, termed "weak zone". The slope is initially in precarious static equilibrium with failure being imminent: at any external triggering the unstable soil will slide along the weak zone evolving in the form of a rotational landslide. During this process, the pipeline that is practically fixed at the stable crest and toe of the slope will undergo significant distress.



Figure 1. The studied problem: (a) pipeline crossing a plane-strain rotationally evolving landslide; (b) view of typical cross-section; (c) geometry of the fitting element at Point O (i.e. at the slope crest).

NUMERICAL METHODOLOGY

Landslide simulation requires modeling of soil of the order of several hundred meters while on the other hand the pipe diameter is of the order of 1-2 meters. In order to model all possible structural instability phenomena (e.g. local bucking, excessive ovalisation of the pipe section etc.), an extremely fine mesh discretization is required. Hence, preserving the same mesh requirements to the surrounding soil would have shaped an enormous mesh of millions of finite elements that is computationally unattractive. To overcome this obstacle, an original two-step procedure is introduced and schematically described in Fig. 2.

In the first step, a "global" model is employed to simulate the evolution of soil movement during the landslide event. The output of this first-step is a set of displacement vectors to be assigned as boundary conditions at the respective soil nodes of the subsequent step. Here, the pipeline presence is tactically ignored – a reasonable assumption for landslides geometries with an out-of-plane dimension significantly larger than the pipeline diameter. Evidently, in this global model the mesh requirements are manageable while a relatively coarse mesh (as that of Fig. 2a) is considered appropriate to model the evolution of the landslide movement. Note that, in plane strain landslides (as the one analyzed herein) a 2-d numerical model would also be just sufficient.

In the second step, an extremely fine "local" model is introduced comprising the pipeline and portion of the surrounding soil. Here, the landslide action is introduced as 3-d displacement vectors (calculated by the "Global" model) assigned at the bottom and lateral nodes of the local model. [To handle the incompatibility between the coarse mesh of the Global model and the fine mesh of the local model, a simple mathematical mapping formulation has been developed]. The pipeline distress is later estimated as a function of the soil-movement amplitude.



Figure 2. Outline of the proposed 2-step numerical methodology: the vectors of soil movement u estimated by the analysis of the GLOBAL Model are assumed at the bottom, and lateral boundaries of the LOCAL model to estimate the induced pipeline distress.

The realistic simulation of the interface condition between the steel pipe and the surrounding soil calls for the use of special-purpose contact elements. The latter allow the pipe to slide on the ground, while the maximum mobilized shear resistance τ_{max} is a function of the frictional coefficient μ . In our study μ has been considered equal to 0.5. As the landslide movement evolves, the pipeline undergoes transverse and longitudinal

displacements and the resistance offered by the surrounding soil steadily increases depending on the soil characteristics. Commonly, this resistance reaches an ultimate peak as the soil reaches failure, while, under strongly dilatant conditions, a stiff initial response up to peak resistance may be followed by softening. Therefore a key aspect of the numerical methodology is the rigorous simulation of the actual soil behavior under large deformations. To this end, an elastoplastic Mohr–Coulomb constitutive model with isotropic strain softening is utilized [Anastasopoulos et al. (2007)]. Strain softening is introduced by reducing the mobilized friction angle φ_{mob} and the mobilized dilation angle ψ_{mob} with the increase of octahedral plastic shear strain to φ_{res} and 0 respectively.



Figure 3. (a) The plane strain free-field (ff) model and (b) snapshots of the deformed mesh with vectors of displacements, at three characteristic instants: triggering, evolution and deposition of the landslide.

LANDSLIDE EVOLUTION ANALYSIS

Fig. 3 presents the numerical model utilized to analyze the landslide. Apart from the inclined part of the slope, an additional length of 30 meters from the crest and 30 meters from the foot of the slope are also incorporated in the model to achieve free field conditions. The healthy soil is modelled with dense sand and the sliding mass is modelled with loose sand (considered to be deposits from a previous landslide triggering).

The constitutive model presented in the previous section is used to describe the soil behavior. The loose sand has a specific unit weight $\gamma_1 = 17 \text{ kN/m}^3$ friction angle of $\varphi_1 = 34^\circ$, dilation angle $\psi_1 = 5^\circ$, Young's modulus $E_1 \approx 4.5$ MPa and it does not exhibit softening behavior. The dense sand is described by a friction angle $\varphi_2 = 45^\circ$, $\varphi_{res} = 37^\circ$, $\psi = (\varphi_{peak} - \varphi_{res})/0.8 = 10^\circ$ and for the simulation of its softening behavior we consider $\gamma_{yield} = 0.01$, $\gamma_{peak} = 0.03$ and $\gamma_{res} = 0.08$. It is assumed to have a specific unit weight $\gamma_2 = 20 \text{ kN/m}^3$ and its elastic behavior is described by Young's modulus of $E_2 \approx 17$ MPa. The sliding surface is simulated with a row of elements of reduced stiffness and strength ($E_r = 3$ MPa and $\varphi_r = 300$). The landslide is triggered by progressively reducing the strength of these elements with the analysis time (the maximum friction angle φ_{max} drops to $\varphi_{res} = 5$ within 1 second).

Fig. 3b presents the results of the free field analysis. The deformed mesh of the numerical model with superimposed displacement contours is used to show the triggering and evolution of the landslide. Due to the decay of the sliding surface strength within the first 1 seconds, the sliding mass starts mobilizing, yet the landslide has not been fully mobilized. As expected, the landslide triggering occurs near the crest of the slope, where the inclination of the sliding surface is steeper. At t = 5 sec the landslide has been fully mobilized with the distribution of the displacement magnitude appearing quite uniform throughout the sliding mass. Finally, the sliding mass decelerates and ultimately stops, creating deposits on the previously flat slope foot.



Figure 4. The local Model : (a) view of the mesh of the soil prism and of the pipeline (b) the cross-section of the analyzed system, (c) a closer view of a typical pipe segment.

PIPELINE RESPONSE SUBJECTED TO LANDSLIDE-INDUCED ACIOTNS

As already mentioned, to investigate the response of the pipeline a second "local" more refined numerical model is employed, that incorporates the pipe. The key aspects of this "local" pipeline – landslide interaction model are presented in Fig. 4. The displacement time histories computed from the "global" free field model are imposed at the bottom and the sides of a soil prism at the vicinity of the pipe. These boundaries are placed at distance 2D from the pipe axis, where D is the pipe diameter. Although the proximity of the boundaries may lead to a slight underestimation of the pipe – landslide interaction, this was a reasonable compromise between accuracy in results and computational efficiency. To accommodate the change in direction, elbow segments are used. In this case the bends that are used are typical examples of cold bending, with a radius of R = 40D (where D is the pipe diameter). The pipe is modelled with four-node reduced integration shell elements, using a total of 48 elements along the circumference of length d_{FE} = 2.5 cm at the areas of significant distress. Finally, to account for the infinite length of the pipe beyond the boundaries of the model, nonlinear springs are adjusted to the boundaries of the 3D model. These springs are calibrated following the methodology presented by Vazouras et al (2015) for infinitely long pipeline.



Figure 5. Pipeline subjected to the rotationally evolved landslide: (a) Deformed pipeline with superimposed Mises stresses (red sections experience most intense bending); (b) schematic illustration of the key stressing mechanisms along the pipe axis; (c) Spatial distribution of axial strains at the top and bottom centerline of

the pipe.

Fig. 5 presents the results of the pipeline – landslide interaction analysis corresponding to the instant when the displacement at the slope toe (point B) equals $U_{toe} = 2.5 \text{ m}$: a quite severe landslide event has occurred. It is evident that the pipe is extensively deformed under the soil actions originated from the rotational movement of the sliding mass. The latter may be analyzed in two components: an axial and a normal to the pipe axis displacement (Fig. 5b). The axial component tends to drag the pipeline downslope, inducing tension near the landslide crown and compression near the landslide toe. Yet, the axial forces transmitted onto the pipeline are bounded by the low strength of the pipe-soil interface (described by a friction coefficient $\mu = 0.5$) and as such the axial distress of the pipe appears not to be critical for the pipeline safety. On the contrary, the normal component of the soil movement causes significant stressing on the pipe. Near the landslide crown (area A') the (perpendicular to the pipe axis) relative soil displacement is negative (i.e. drags the pipe downwards), while close to the toe of the landslide, the soil moves upwards (u_{s,2a} >0) carrying the pipe along. Since the pipeline is not free to follow the soil movement but is rather fixed at the crest and at the tow within the stable ground, it unavoidably bends; bending is positive at Area A' and negative at Area A. Moreover, since the subgrade pressure is much greater than that produced by the overlying soil (i.e. the downward soil reactions are much higher than the upward reactions), the pipeline is more distressed near the toe of the landslide in Area A. The combination of bending and compression near the toe of the landslide ultimately leads to local buckling of the fitting component. The spatial distribution of ε_{xx} along the pipeline length is depicted in Fig. 5c. In Area A (pipe segment that is under prevailing bending near the slope crest) and in Area B (the pipe segment that is mainly under axial loading) the developed strains are limited and the pipe responds elastically even for such a large soil displacement. In stark contrast, near the toe of the slope the landslide-induced actions lead to severe distress of the pipeline. In Area A' (extending approximately from x = 0 to x = 15 m) the pipe develops excessive tensile and compressive strains at the top and bottom side respectively as it bends due to the soil movement. Equally adverse is the behavior of the pipe segment that is effectively anchored within the stable soil (extending approximately from x = -10 to x = 0 m), that develops extreme compressive strains on the top side due to reversed bending. As attested by the analysis, the critical section of the pipe where local buckling initiates first, is in latter area, where the pipe is anchored within the stable soil.



Figure 6. (a) Variation of axial strains at the critical area of the pipeline before and after the formation of local buckle. (b) Axial strain evolution at the compressive side of the critical pipeline area.

The evolution of local buckling at the critical section of the elbow is graphically depicted in Fig. 6. Fig. 6a portrays the critical pipe segment before and after buckling. The initially uniformly distributed compressive strains (top figure) have localized and formed a distinctive wrinkle at the pipe wall (bottom Figure). Fig. 6b plots the progressive growth of axial strains as a function of landslide movement. Note that for landslide displacement of the order of $U_{toe} = 1.1$ m strains start localizing forming a wave-like pattern. As the

displacement increases, the bucking progresses rapidly: due to pipe wall wrinkling, bending strains develop, followed by significant tensile strains at the "ridge" of the buckle. As a result, the longitudinal compressive strains at the outer surface of the pipe wall start decreasing, forming a short wave. At that instant bucking has fully shaped [Vazouras et al (2010)].



Figure 7. Evolution of the maximum axial strain along the pipeline with increasing U_{toe} .

To quantify the safety margins until buckling initiation Fig. 7 presents the minimum axial strain along the pipe (that is located in the critical section) with the evolution of the landslide as described by the displacement of the landslide toe. Observe the profound change in the rate of accumulating compressive strain for displacement $U_{toe} = 1.1$ m. From that point on, the pipe accumulates strain with a very large rate, signifying the occurrence of an instability phenomenon (local buckling).

CONCLUSIONS

In this paper an original finite element methodology is proposed that allows the study of the coupled soilpipeline problem in a 2-Step Procedure. During the first Step, a "global" relatively coarse model is employed to simulate the soil movement evolution during the landslide event. The output of this first-step is a set of displacement vectors that are assigned as boundary conditions at the respective soil nodes of an extremely fine "local" model which is able to capture all modes of pipeline failure and is incorporated during the second Step. The methodology is employed to investigate the structural performance of a buried steel pipeline subjected to a plane-strain rotationally evolving landslide. It is concluded that, during the landslide event, the pipeline undergoes two key stressing mechanisms: (i) a pull-out action that tends to drag the pipeline downslope, inducing tension near the landslide crown and compression near the landslide toe and (ii) a lateral push at the area of the landslide toe that induces bending on the pipe. The combination of bending and compression near the landslide toe causes the failure of the fitting component (elbow) due to local buckling, highlighting the vulnerability of the pipeline in this specific area.

REFERENCES

American Lifelines Alliance. Guidelines for the design of buried steel pipes. ASCE, New York (2001).

- Anastasopoulos I., Gazetas G., Bransby M.F., Davies M.C.R., El Nahas A. (2007), "Fault rupture propagation through sand: finite element analysis and validation through centrifuge experiments", *Journal of Geotechnical and Geoenvironmental Enineering*, ASCE, 133(8): 943–958.
- Cocchetti, G., Prisco, C., Galli, A., and Nova, R. (2009). "Soil-pipeline interaction along unstable slopes: a coupled three-dimensional approach. Part 1: Theoretical formulation", *Can. Geotech. J.*, 46:1289-1304.
- Daiyan, N., Kenny, S., Phillips, R., and Popescu, R. (2009). "Parametric study of lateral-vertical pipeline/soil interaction in clay, *1st Int./1st Eng. Mechanics and Materials Specialty Conf.*, St. John's, NL, Canada.

- Daiyan, N., Kenny, S., Phillips, R., and Popescu, R. (2010). "Numerical investigation of oblique pipeline/soil interaction in sand", 8th Int. Pipeline Conf., Calgary, Alberta, Canada.
- Gantes, C.J., Bouckovalas, G.D., and Koumousis, V.K., "Slope Failure Verification of Buried Steel Pipelines", 10th International Conference on Applications of Advanced Technologies in Transportation, Athens, Greece, May 27-31, 2008.
- Guo, P. (2005). "Numerical modeling of pipe-soil interaction under oblique loading." J. of Geotech. and Geoenviron. Eng., ASCE, 131(2): 260-268.
- Hamada, M. and O'Rourke, T.D., Eds., (1992). "Case Studies of Liquefaction and Lifeline Per-formance During Past Earthquakes", Vol. 1, NCEER-92-0001, National Center for Earth-quake Engineering Research, State University of New York at Buffalo, NY.
- Hsu T.W., Chen Y.J., and Hung W.C. (2006) "Soil resistant to oblique movement of buried pipes in dense sand." *Journal of Transportation Engineering*, 132(2), 175-181.
- Jibson, R.W. and Keefer, D.K., (1993), "Analysis of the Seismic Origin of Landslides: Examples from the New Madrid Seismic Zone," *Geological Society of America Bulletin*, April, Vol. 105, pp. 521-536.
- O'Rourke, M. J., & Liu, X. (1999). Response of buried pipelines subject to earthquake effects.
- Phillips, R., Nobahar, A., and Zhou, J. (2004). Combined axial and lateral pipe-soil interaction relationship, 5th Int. *Pipeline Conf., Calgary*, Alberta, Canada.
- Pike, K., Seo, D., and Kenny, S. (2011). "Continuum modelling of ice gouge events: Observations and assessment", Arctic Technology Conf., Houston, TX, USA.
- PRCI, (2004), "Guidelines for the Seismic Design and Assessment of Natural Gas and Liquid Hydrocarbon Pipelines, Pipeline Design, Construction and Operations", Edited by Honegger, D. G., and Nyman D. J., Technical Committee of Pipeline Research Council International (PRCI) Inc, October 2004
- Vazouras P, Karamanos SA, Dakoulas P. Finite element analysis of buried steel pipelines under strike-slip fault displacements. *Soil Dynamics and Earth- quake Engineering* 2010;30(11):1361–76.
- Vazouras, P., Dakoulas, P., & Karamanos, S. A. (2015). Pipe-soil interaction and pipeline performance under strikeslip fault movements. *Soil Dynamics and Earthquake Engineering*, 72, 48-65.
- Yimsiri, S., Soga, K., Yoshizaki, K., Dasari, G.R., and O'Rourke, T.D. (2004). "Lateral and upward soil-pipeline interactions in sand for deep embedment conditions", J. of Geotech. and Geoenviron. Eng., ASCE, 130(8): 830-842.