

A study on the effects of slope grade on the soil-pipeline interaction loading

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ABSTRACT

In this work, a numerical model is calibrated using the force-displacement data of Trautmann et al. (1985). The numerical technique is implemented to study the effects of slope angle on soil-pipeline interaction loading. For this purpose, the pipe is displaced in horizontal, upward and oblique directions and force-displacement curves are plotted for each analysis. The results show that the existing p-y curves of soil-pipeline interaction available in the design guidelines might be unconservative when the pipe is buried in a slope. The soil displacement contours, plotted for each analysis, demonstrate the mechanism of soil failure against the pipe displacement. This preliminary numerical study highlights the importance of updating the force-displacement curves of soil-pipeline interaction to take into consideration the slope inclination.

RÉSUMÉ

Dans cet article, un modèle numérique est calibré en utilisant des données de force-déplacement de Trautmann et al. (1985). La technique numérique est implémentée pour étudier les effets de l'inclinaison des pentes sur le chargement de l'interaction sol-pipeline. À cette fin, le tuyau est déplacé dans les directions horizontale, ascendante et oblique et des courbes de force-déplacement sont tracées pour chaque analyse. Cette analyse numérique montre que les courbes actuelles de p-y de l'interaction sol-pipeline sont non-conservatrices lorsque le tuyau est enterré dans une pente. Les contours de déplacement du sol, tracés pour chaque analyse, démontrent clairement le mécanisme de rupture du sol contre le déplacement du tuyau. Cette étude numérique met en évidence l'importance de la mise à jour des courbes force-déplacement de l'interaction sol-pipeline afin de prendre en compte l'inclinaison de la pente.

1 INTRODUCTION

Soil - pipeline Interaction is one of the very cutting-edge engineering topics, which is still receiving significant attention from industry and academia due to its relatively high uncertainty and complexity. Researchers have contributed much to the understanding of soil-pipeline interaction; however, there are still considerable ambiguities regarding this mechanism — for example, the ground surface inclination which may be a significant factor, has received limited attention. O'Rourke and Trautmann (1985), in their original formulations, assumed that the pipe is buried under a flat ground surface for pull-out testing. This is a bounding simplified assumption that only represents a portion of those cases in practice. There are many examples of pipelines that are traversing valleys, hills, and unstable slopes. For example, a study by Ferreira (2016) showed that there are 35 cases of pipes buried in moderate to high-risk slopes just in a gas pipeline network in Manitoba. Therefore, the question remains whether the current design guidelines, which were developed based on the pull-out tests in flat ground surface, are applicable for the pipelines buried in an inclined slope. If one looks back through the histories of pipeline failures subjected to earthquakes and landslides, they are generally unconservative.

Katebi et al. (2019) showed an example of pipeline analysis buried in the slope. The pipeline in Harrowby, MB, crossing a 90-m-deep valley, is subjected to slow ongoing landslide movements. The interaction loading on the

Harrowby pipeline had been increased slowly over time due to the slow land movement until it reaches the ultimate soil-pipe interaction capacity. The presented analysis was according to the existing pipeline design guideline, PRCI (2017). However, the current pipeline design guidelines, ALA (2005) and PRCI (2017), do not consider the effects of slope grade on the soil-pipeline interaction loading — the soil interaction loading parameters are based on experiments in which the slope grade was assumed to be zero.

The present study shows the importance of considering the slope grade on soil-pipeline interaction loading through numerical modelling with Abaqus Finite Element (FE) software. Use of unconservative parameters in estimating the soil-pipeline interaction loading in the case of the Harrowby pipeline, for example, might lead to the unexpected yielding of the pipeline. Therefore, the significance of studying the soil-pipeline interaction loading for pipes crossing valleys and slopes is important.

2 NUMERICAL ANALYSIS

A preliminary study is carried out using ABAQUS/Standard FE software. The result is calibrated using the force-displacement data of Trautmann et al. (1985). The calibrated numerical technique is used to study the effects of slope grade on lateral, oblique and uplift soil-pipeline interaction. The geometry used for the numerical analysis is shown in Figure 1, where H/D is the burial depth ratio, H is the pipe burial depth, D is the pipe diameter; β is the slope angle; α is the direction of pipe movement relative to the vertical line.

The numerical analysis is first conducted for lateral soil-pipeline interaction for a 102-mm-diameter pipe buried 1.5D below the ground surface, where the surface is at a slope of 0°, and 20° angle. The effects of slope grade on oblique soil-pipeline interaction are also presented for 36 combinations of H/D , α and β as summarized in Table 1.

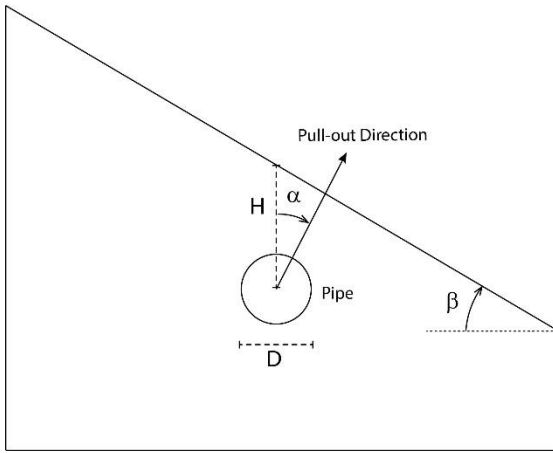


Figure 1. Schematic view of a buried pipe in a slope

Table 1- Summary of the analyses conducted in the present study

Parameter	D					
H/D	1.5	4				
α	0°	45°	-45°			
β	0°	10°	15°	20°	25°	30°

2.1 Soil Parameters

In the present study, the classical elastoplastic Mohr-Coulomb model is used to represent the soil. The soil properties are selected to be representative of the characteristics of Cornell University (CU) Filter Sand used by Trautmann et al. (1985). The selection technique of these parameters is provided in the following subsections (Jung et al. 2013):

- plane strain peak friction angle, ϕ'_{ps-p}
- peak dilation angle, ψ_p
- The secant soil modulus corresponding to the pipe deformation at the pipe peak load for each burial depth, E_{α} .

2.1.1 Plane Strain Internal Friction Angle

The present numerical analysis is conducted under plane strain condition, and therefore, it is necessary to define the

plane strain angles of internal friction and dilation (Jung et al. 2013).

A relationship first described by Davis (1986) is used to correlate plane strain peak friction angle (ϕ'_{ps-p}) to the direct shear peak friction angle (ϕ'_{ds-p}):

$$\sin \phi'_{ps-p} = \frac{\tan \phi'_{ds-p}}{\cos \psi_p + \sin \psi_p \tan \phi'_{ds-p}} \quad [1]$$

The parameters, ϕ'_{ds-p} and ϕ'_{ps-p} , are required to find the plane strain friction angle from Equation 1. Numerous direct shear (DS) tests were carried out by Olson (2009) to determine the peak internal friction angle and the peak dilation angle of CU sand used in the experiments of Trautmann et al. (1985). The result of their work is reproduced in Figure 2.

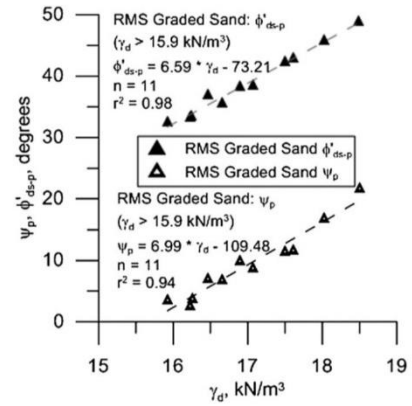


Figure 2. ϕ'_{ds-p} and ψ_p versus γ_d for CU sand (after Olson 2009)

For medium dense CU sand with the dry density of 16.4 kN/m^3 , DS peak friction angle is 36°, and peak dilation angle is 4°. Using Equation 1, the plane strain peak friction angle of the CU sand is equal to 44°.

2.1.2 Secant Soil Modulus

Young's modulus is a critical material property when applying an elastoplastic model in a numerical analysis. The Young's modulus of soil should be compatible with the strain in the soil under the peak soil-pipeline interaction loading. Jung et al. (2013) estimated the secant modulus of CU Filter sand corresponding to the stress level of the peak pipe load through several finite element analyses using the initial tangent Young's modulus of soil as mentioned in Table 2.

Table 2- Secant soil modulus for upward soil-pipeline interaction loading (after Jung et al. 2013)

Soil	H/D	E_{α} (kPa)
Medium CU sand	1.5-13	400-2800
Very dense CU sand	1.5-13	700-5200

In the present study, the secant soil modulus of 920 kPa is used for the burial depth of 4, and 400 kPa for the burial depth of 1.5.

2.2 Finite Element Modelling

ABAQUS/Standard is used for this preliminary numerical study. The pipe is considered rigid, so a plane strain model is suitable for the analysis. Eight-node biquadratic plane strain quadrilateral element with full integration method is used for soil. The mesh is generated using a quad element shape with the approximate size of 5 cm. An advancing front algorithm is used for mesh generation. A mapped meshing algorithm is implemented to enhance the quality of the mesh on faces of the model with simple geometry. The geometry of the model is divided into appropriate partitions, which facilitates the meshing process. Figure 2 shows an example of the generated mesh. No mesh sensitivity analysis is performed in this work.

The soil-pipeline interface is modelled using the surface-to-surface contact discretization. The outer pipe surface is defined as the master surface and the soil surface as the slave surface. A finite sliding formulation is used in the interface to allow an arbitrary and free sliding of the surfaces. The behaviour of the interface is modelled using a tangential and normal interaction. For normal contact, a pressure-overclosure of hard contact, which uses the classical Lagrange multiplier method of constraint enforcement, is implemented. The separation after contact is allowed for the normal behaviour of the interface. A penalty friction formulation is used for tangential behaviour. The friction models in ABAQUS includes the classical isotropic Coulomb friction model. This model can define friction in terms of slip rate and contact pressure. However, for simplicity, the contact behaviour is considered independent of the slip rate and contact pressure in this study — an elastic slip of 0.005 fractions of the characteristic surface dimensions used in the analysis. Jung et al. (2013) reported that the interface friction angle between soil and pipe in the experiments of Trautmann (1985) is $0.6\phi_{ds-p}$. The plane strain soil friction angle is 44° , so the soil-pipe interface friction angle would be 26.4° that gives us the friction coefficient of 0.5 for the numerical analysis. To avoid convergence problems and improve the numerical stability, a cohesion of 0.3 kPa is used in all simulations.

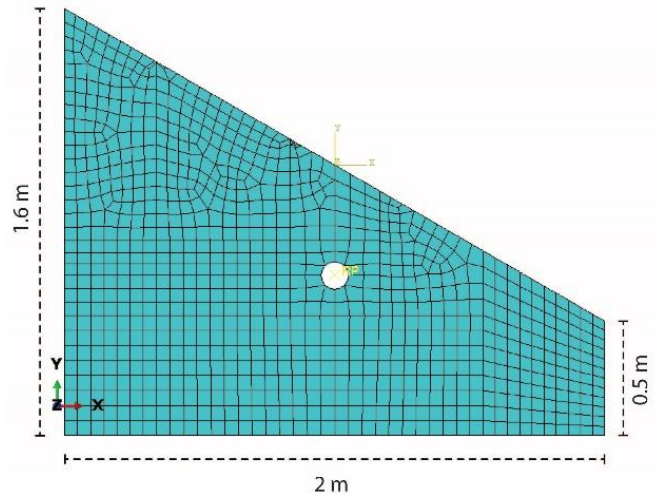


Figure 2. An example of the meshed model

2.3 Validation

The result of the simulation is first calibrated using the experiments of Trautmann et al. (1985) for upward loading for H/D of 1.5 and 4, as shown in Figures 3 and 4, respectively. The FE analysis reasonably estimates the peak uplift dimensionless force although the stiffness is poorly captured. The result of the experiments shows a rapid drop of load after the ultimate loading, while the FE analysis does not illustrate consistent behaviour. This inconsistency is expected when an elastoplastic MC model is used in the simulation. Previous researchers also found a similar elastic perfectly plastic force-displacement response for the MC model (Yimsiri et al. 2004; Roy et al. 2016). The rapid reduction of the uplift force with upward pipe displacement occurs due to the combined effects of three factors: (1) decreases in size of the failure wedge, (2) reduction of shear resistance with plastic shear strain, and (3) reduction of cover depth (Roy et al. 2018). This behaviour can be predicted through implementing a strain-softening soil constitutive model, e.g. Anastasopoulos et al. (2007), Roy et al. (2016).

The slope of the elastic part of the p-y curve is also an essential factor for defining soil spring parameters, which is not adequately predicted by this FE analysis (See Figures 3 and 4). The elastic modulus of the soil can be better defined using a strain-dependent elastic modulus. Furthermore, the prediction of the elastic behaviour of the contact needs improvement, which can be achieved using more accurate contact behaviour. This numerical analysis can be improved for an extensive parametric study that can be automated using Python scripting in ABAQUS. However, this is left for future study. The peak dimensionless force is the primary concern for the present study and Figures 3 and 4 show that the present modelling technique using the MC model can reasonably predict the peak dimensionless force.

The present numerical analysis is calibrated only with the uplift force-displacement data of Trautmann et al. (1985) and is used for lateral, oblique and uplift soil-pipeline interaction loading. It can be argued that soil is an anisotropic material, and therefore, each loading direction should be calibrated against the corresponding existing

data in that direction. Due to the space limitation, the present numerical model has only been calibrated using the uplift pull-out tests of Trautmann et al. (1985). However, Figure 5 shows that the peak lateral force, estimated by the present numerical modelling technique, is also comparable to the peak lateral force obtained by Trautmann et al. (1985).

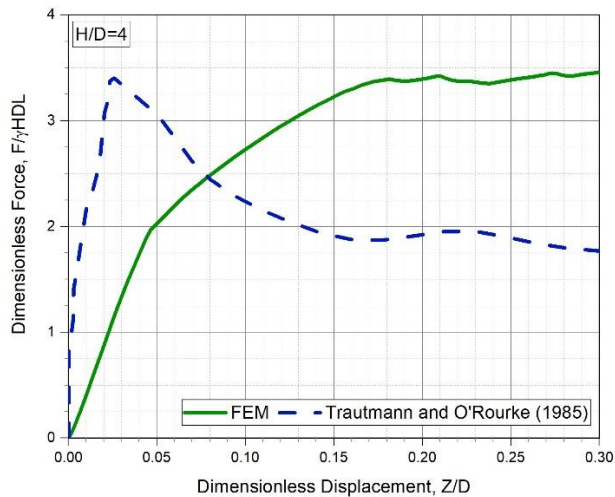


Figure 2. The force-displacement curve for uplift loading. FEM versus Trautmann et al. (1985), $H/D=1.5$

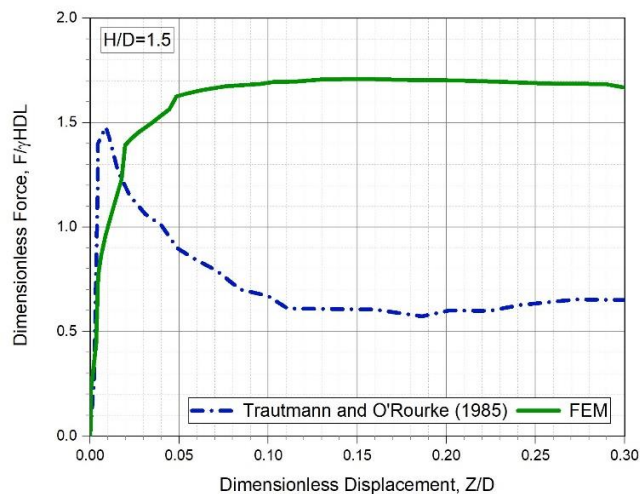


Figure 4. The force-displacement curve for uplift loading. FEM versus Trautmann et al. (1985), $H/D=4$

3 RESULTS

In this section, the result of the analysis is presented to highlight the effects of slope grade on the lateral and oblique soil-pipeline interaction loading.

3.1 Lateral Soil-Pipeline interaction

Figure 5 presents the force-displacement curve of a 102-mm-diameter rigid pipe buried with $H=1.5D$. The pipe

displacement over 5 cm is simulated in a horizontal direction under three different conditions:

- $\beta=0^\circ$ — the dark grey curve in Figure 5 shows the result for horizontal lateral loading when the slope is zero.
- $\beta=20^\circ$ — the green curve in Figure 5 shows the result for horizontal lateral loading to the left (upslope) when the slope is 20° .
- $\beta=20^\circ$ — the red curve in Figure 5 shows the result for horizontal lateral loading to the right (downslope) when the slope is 20° .

The results show that increasing the slope grade from 0° to 20° significantly increases the horizontal lateral peak force on the pipe when the pipe is moved upslope ($\alpha=90^\circ$) (See Fig. 5). On the other hand, the horizontal lateral peak force is substantially reduced when the pipe is moved downslope ($\alpha=90^\circ$) increasing the slope from zero to twenty degrees in the same configuration.

The green curve in Figure 5 represents the soil-pipeline interaction in the case of a landslide because the soil pushes the pipe against the slope as it moves downhill. This phenomenon suggests that the peak force presented by the current design guidelines underestimates the lateral soil-pipeline interaction loading when the pipe is buried in a slope. The soil failure mechanisms, as shown in Figure 6, 7, and 8 for the three abovementioned cases, explain why the peak forces are dependent on the slope angle. The zone of disturbed soil in front of the pipe in Figure 7, as shown by the light blue colour, is more significant than that shown by the same colour in Figure 6. Because the restraint is coming from a larger volume of soil in front of the pipe, a higher peak force is observed. Note that the pipe is displaced in the horizontal direction in all cases. A finite sliding formulation is used for the interface, and as such, the pipe is free to move in any arbitrary direction as it is pulled horizontally. Figures 6, 7 and 8 show that the pipe has been displaced in the oblique direction although the loading direction was completely horizontal. The reason for this oblique movement of the pipe is the oblique reaction forces of the soil on the bottom of the pipe. In other words, the pipe tends to move towards the ground surface where no reaction force is exerted on the pipe. This mechanism clearly shows the impact of the ground surface on the soil-pipeline interaction.

The soil-pipeline interaction is a half-space problem; the further away the pipe is from the ground surface, the higher the soil restraint would be. A sloped ground substantially increases the soil resistance to the pipe movement as there is more soil on the upside of the pipe to resist the pipe movement towards that direction. If the pipe is buried deep enough that the soil failure mechanism is not extended to the ground surface, the effects of the ground slope could be ignored. As a result, the significance of the slope grade on the p-y curves diminishes by increasing the pipe burial depth.

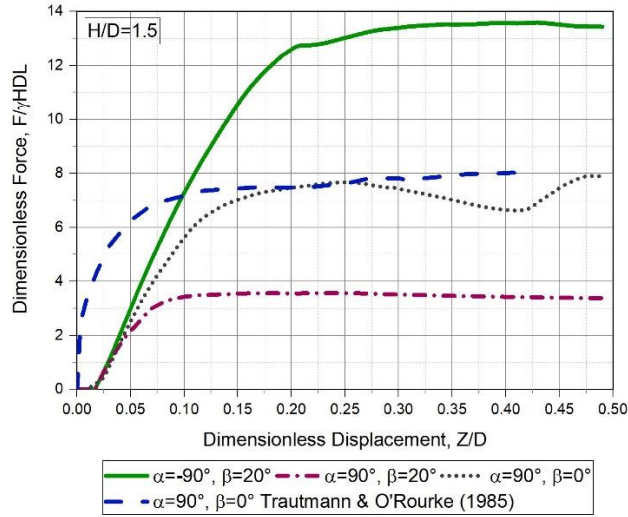


Figure 5. Force-displacement curves of pipe pulled horizontally in the soil; α and β are defined in Figure 2

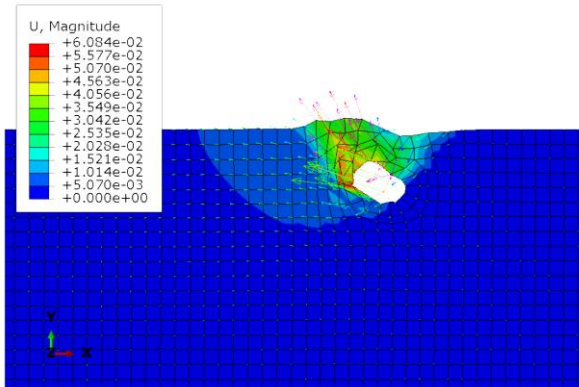


Figure 6. Contours of soil deformation subjected to horizontal pipe displacement; $\alpha = -90^\circ$, $\beta = 0^\circ$, $H/D = 1.5$

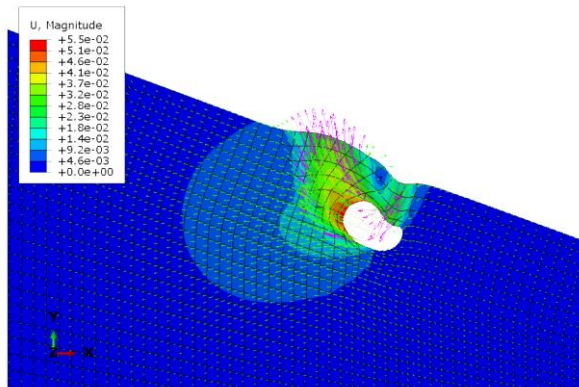


Figure 7. Contours of soil deformation subjected to horizontal pipe displacement; $\alpha = -90^\circ$, $\beta = 20^\circ$, $H/D = 1.5$

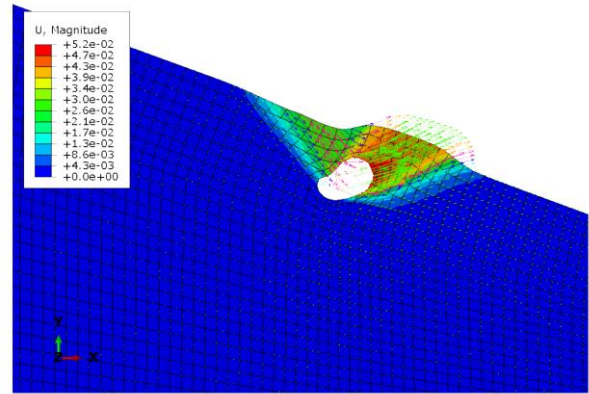


Figure 8. Contours of soil deformation subjected to horizontal pipe displacement; $\alpha = 90^\circ$, $\beta = 20^\circ$, $H/D = 1.5$

3.2 Oblique Soil-Pipeline Interaction Loading

Figure 9 shows the force-displacement curves obtained for oblique loading ($\alpha = 45^\circ$). The peak oblique force when the slope grade is 10° is significantly different from that of 0° slope grade. The oblique force when the pipe movement is oblique to the left (against the slope) is significantly higher compared to that to the right (toward the slope). Furthermore, for the oblique pipe movement to the left (upslope), the resisting force has not been reached to the peak even after the dimensionless displacement of 0.6 as shown by the green curve in Figure 9. The oblique movement to the right (downslope) results in lower peak force. Increasing the slope angle would increase the peak load when the pipe is pulled to the left and decrease the peak load when the pipe is pulled to the right. Many studies in the literature examine the peak oblique soil-pipeline interaction loading disregarding the slope grade. This preliminary study identifies the importance of defining the peak soil-pipeline interaction loading in combination with the slope grade. This study suggests that the current p-y curves underestimate the peak soil-pipe interaction loading for pipes buried in a slope in the case of land moving, i.e. in a landslide, the pipeline is subjected to an oblique movement to the upslope because the soil movement is to the downslope.

Comparing the blue and the dark grey curves in Figure 10, it can be learned that the soil restraint against pipe displacement is higher for downslope pull-out when the slope is zero and it reduces by increasing the slope to 20° .

Figure 11 shows the effects of the slope angle on the oblique force when the pipe displacement is to the right (downslope); increasing the slope grade results in a reduction of the peak force.

Figure 12 presents the effects of the slope grade on the oblique force when the pipe displacement is to the left (upslope); increasing the slope grade results in an increase of the peak force. This increase is most significant when the slope grade is increased from 0° to 10° .

The contours of soil deformation subjected to the oblique pipe displacement are shown in Figures 13, 14, and 15. The significant difference of soil deformation contours in these figures could be used to corroborate further why the peak force of oblique pipe displacement to the left (upslope) is much higher than that to the right (downslope). Furthermore, the contours of soil deformation subjected to upward pipe movement are not symmetric; this is expected as the soil is not symmetric relative to the pipe (Figure 15). As expected, the contours of the soil deformation are symmetric when the slope angle is zero (Figure 16). Such a difference in soil failure mechanism reiterates the importance of including slope grade to calculate the soil resistance force on the pipe.

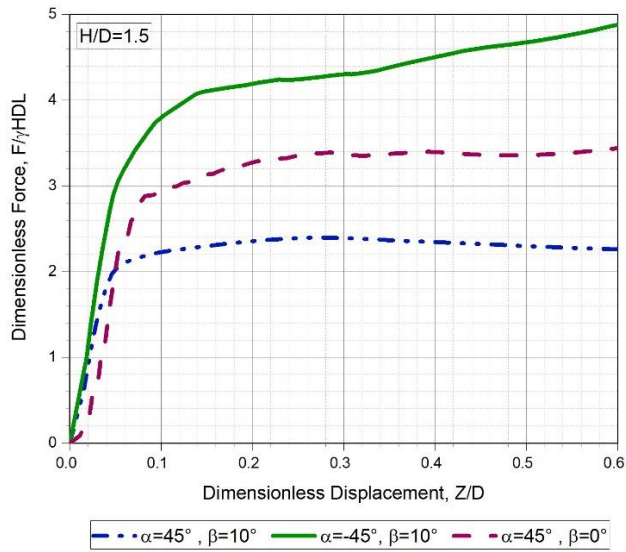


Figure 9. The force-displacement curves of pipes pulled horizontally in the soil, α and β are defined in Fig. 1

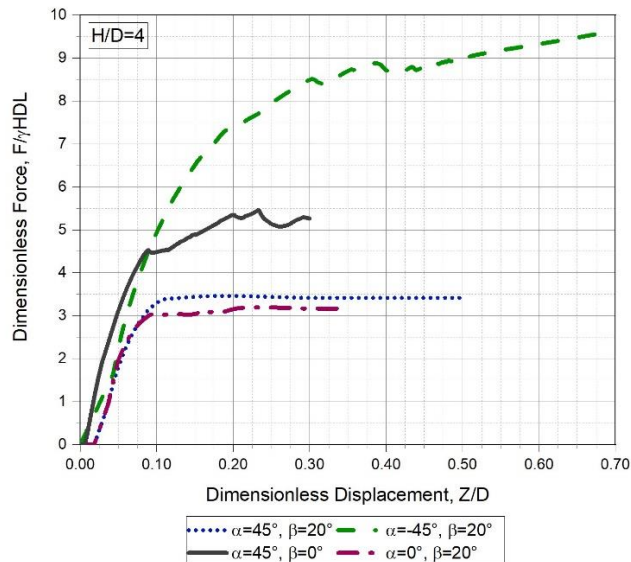


Figure 10. The force-displacement curves of pipes pulled in different directions; α and β are defined in Fig. 1

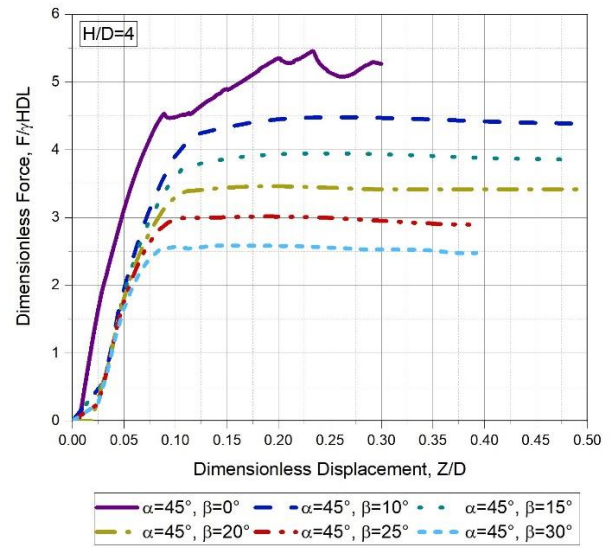


Figure 11. The force-displacement curves of pipes pulled in oblique upslope direction; α and β are defined in Fig. 1

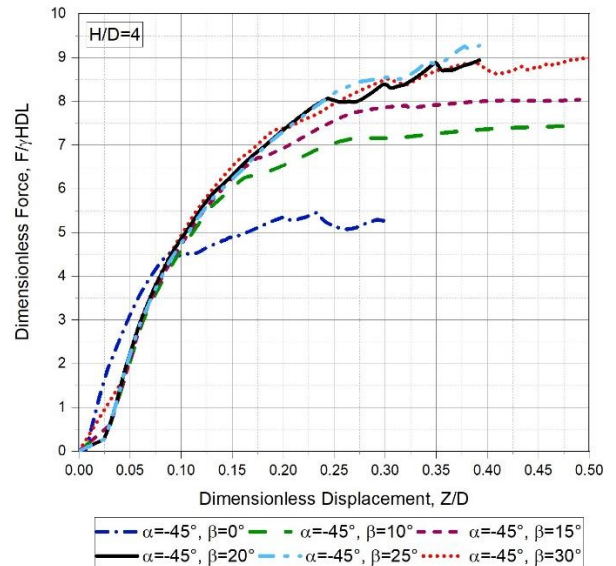


Figure 12. The force-displacement curves of pipe pulled in oblique downslope direction; α and β are shown in Fig. 1

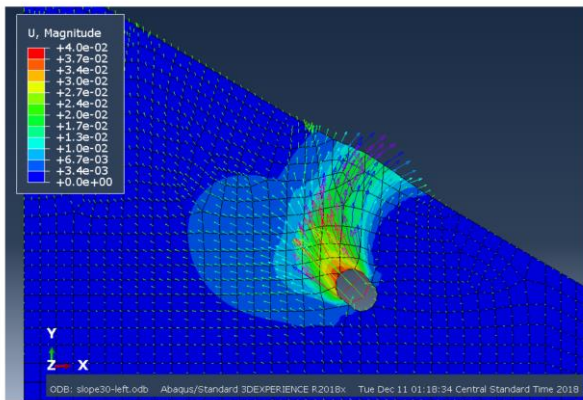


Figure 13. Contours of soil deformation subjected to oblique upslope pipe displacement; ($\alpha=-45^\circ$, $\beta=30^\circ$, $H/D=4$)

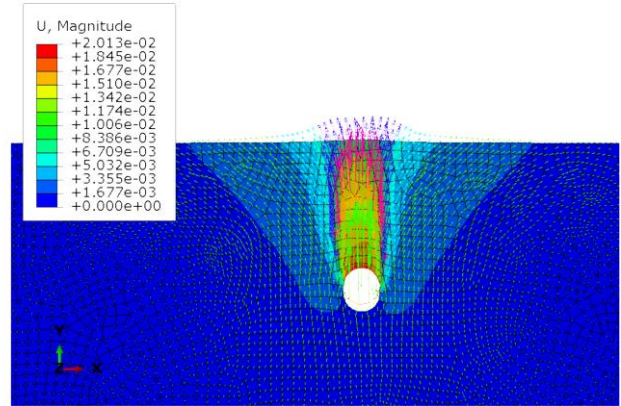


Figure 16. Contours of soil deformation subjected to uplift pipe displacement; ($\alpha=0^\circ$, $\beta=0^\circ$, $H/D=4$)

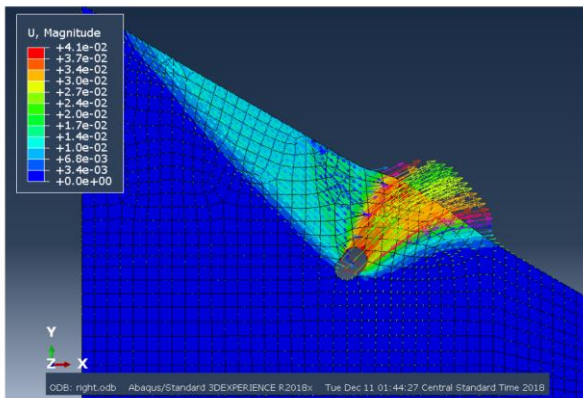


Figure 14. Contours of soil deformation subjected to oblique downslope pipe displacement; ($\alpha=45^\circ$, $\beta=30^\circ$, $H/D=4$)

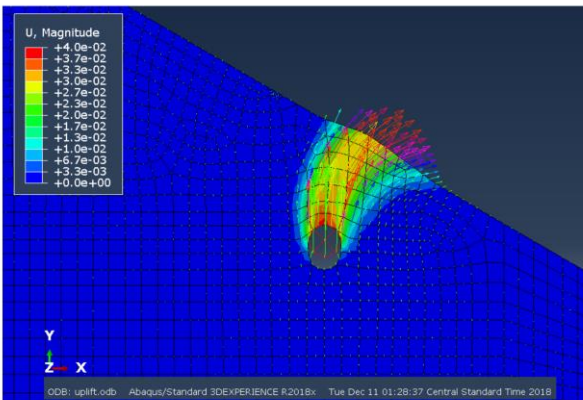


Figure 15. Contours of soil deformation subjected to uplift pipe displacement; ($\alpha=0^\circ$, $\beta=30^\circ$, $H/D=4$)

4 CONCLUSION

This paper presents a preliminary analysis to show the effects of slope grade on soil-pipeline interaction loading. Through a series of finite element simulation, it is demonstrated that the interaction loading on the pipe in a landslide can be significantly larger as compared to those recommended in the existing pipeline design guidelines. This preliminary study highlights the necessity of thorough systematic research to gain further knowledge on the behaviour of pipelines subjected to landslides. As described at length, the dimensionless interaction loading parameters presented by the existing guidelines can be unconservative for pipes crossing valleys and slopes. The authors are further investigating the problem using the full-scale Advanced Soil Pipe Interaction Research (ASPIRe™) facility at the University of British Columbia. The results of the experiments will enhance the understanding of this complex soil-pipe interaction problem and in turn contribute to increasing pipeline safety against geohazards in the sloping ground, primarily located in landslide-prone regions.

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