Application of the SPH Finite Element Method to Evaluate Pipeline Response to Slope Instability and Landslides

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1 Abstract

Buried pipelines operating on active slopes can be subject to lateral and axial loads resulting from slope instability and landslides. The techniques to predict pipeline displacements, loads, stress or strains are not well described in design standards or codes of practice. Finite element analysis based soil-pipe interaction simulation has developed in recent years and is proving to be a useful tool in evaluating the pipeline behavior in response to slope movement.

A description of the BMT pipe soil interaction modeling techniques, their validation against full scale trails and comparison to spring support models has been previously published. This paper describes the modeling techniques and demonstrates the application and versatility of the LS-DYNA 3D continuum SPH (Smooth Particle Hydrodynamic) model to evaluate pipeline behavior and pipeline strain demand. The effects of key parameters, including soil movement mechanism, pipeline geometry, material grade and soil conditions and properties are considered.

The application and results presented in this paper are used to illustrate an advanced soil structure interaction numerical simulation technique combining the LS-DYNA nonlinear SPH formulation with Lagrangian formulation while satisfying all the principles of continuum mechanism.

Keywords: LS-DYNA, SPH, soil-pipe interaction, slope movement, landslide, strain demand, strain relief, geo-hazards

2 Introduction

The response of oil and gas pipelines to permanent ground movement is an important consideration in pipeline route selection, design and in-service integrity management. Ground movement can introduce substantial axial and bending strains on buried pipelines. These strains depend on the force imposed on the pipeline by the interaction of the pipeline and the movement of the surrounding soil.

This paper describes ongoing work involved in a study investigating the mechanical behavior of buried pipelines interacting with active soil movements. Detailed pipe-soil interaction analyses were completed with a 3D continuum Smooth Particle Hydrodynamic (SPH) method. This paper describes LS-DYNA numerical modeling process, its validation against full scale trails and its application to site-specific conditions illustrating the insights produced to support geotechnical hazard pipeline integrity management.

3 Finite Element Model Description

LS-DYNA 971 [1] was used to undertake 3D continuum modelling using the SPH method. Only summary information is presented here regarding the model as it has been described previously [2, 3, 4]. The model had two main components:

1. **The soil** – the soil was modelled using a double hardening plasticity model, where the deviatoric and volumetric yield surfaces are independent. In LS-DYNA, this constitutive model is known as material model 5, or the soil and foam model (MAT_SOIL_AND_FOAM).

2. **The pipeline** - the pipe was modeled using Belyschko-Tasy shell elements with five (5) integration points through the thickness. The pipe was modeled using material 24, “MAT_PIECEWISE_LINEAR_PLASTICITY”.

4 Validation of the SPH Model

The objective of this section is to demonstrate that the response of the pipeline subjected to abrupt ground deformation (surface faulting, lateral spreading, slope failure or landslide) can be simulated using SPH techniques within the non-linear explicit code, LS-DYNA [1]. The BMT soil-pipe interaction models are validated and calibrated based on a wide range of available published full-scale experimental data including pipelines subjected to: Lateral ground movement, Axial ground movement, and Abrupt ground deformations similar to surface faulting. Only a small sample overview of the validation program is presented in this paper.

4.1 Pipeline Subjected to Lateral Movement

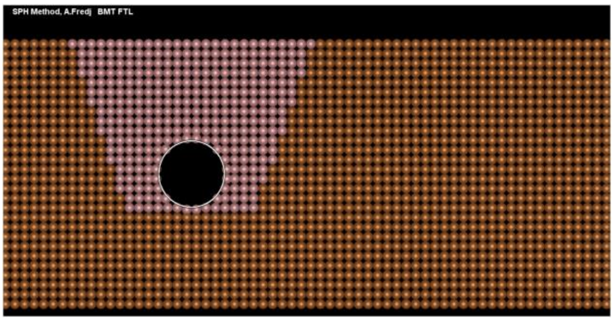
4.1.1 Large Scale Experiment

The SPH pipe-soil intercation model was validated and calibrated based on Karimian et al. [5] large tank experimetal test. The pipe loading tests were performed in a large sand chamber of 2.50m (8.2 ft) x 3.8m (12.5ft) x 2.5m (8.2ft) at the University of British Columbia. Figure 1 (a) shows the experimental setup. The UBC pipe had an outside diameter of 457mm (18 in) and a wall thickness of 12.7mm (0.5in). The pipe was buried at the overburden depth to pipe diameter ratio (H/D) of 1.92in a sand with friction angle of 32^0 and cohesion, c=0kPa.

4.1.2 Finite Element Model

The SPH model used for the analysis is shown in Figure 1(b), with the same geometry as the experimental setup. The soil mass in the tank is taken to be stress free and have uniform properties in its initial state. The displacment is imposed in two steps. In the first step, gravitational acceleration is applied; allowing sufficient time for the soil stresses reach a stationary state. In the second step, the pipe was pulled in the lateral direction by imposing displacement boundary conditions. The pipe and surrounding soil interaction is modeled by Lagarangian contact allowing slip and separation.

(a) UBC Experimental Trial.



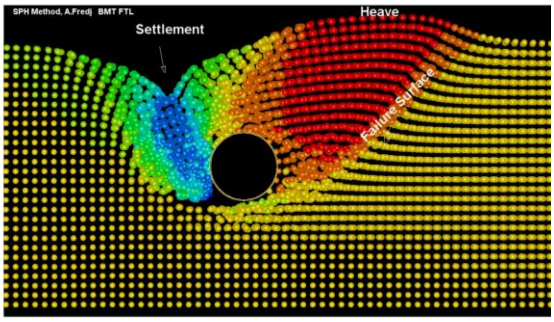
(b) Illustration of the FE Model.

Fig.1: Experimental Trial and FE Model

4.1.3 Comparison of Analytical and Experimental Results

A typical graphical SPH model output is presented in Figure 2 illustrating the simulated displaced position of the pipe and the soil. The figure also shows the shear rupture or slip surface through the soil in the front of the pipe. Qualitatively, the displaced soil profile and flow history make intuitive sense and match the observed behaviors.

Figure 3 presents a comparison of experimental and numerical force-displacement curves demonstrating that for a lateral pipe movement, at H/D of 1.92, the model results match the experimental data well. For both experimental and numerical force displacement using SPH methods, the failure load is about 50kN/m obtained at the pipe displacement of 75mm.



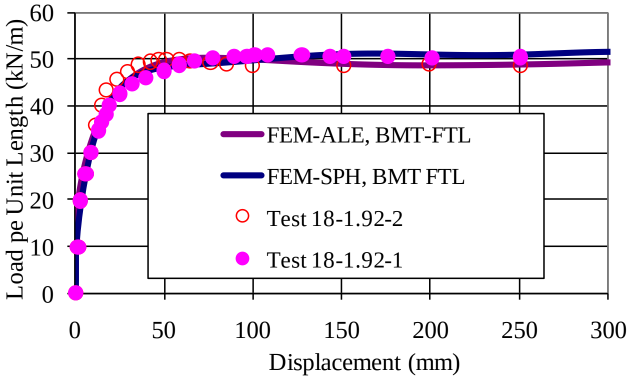


Fig.2: SPH-FEM Simulation – Vertical Displacement.

Fig.3: SPH-FEM Simulation – Comparison Between Numerical Model and Test Results.

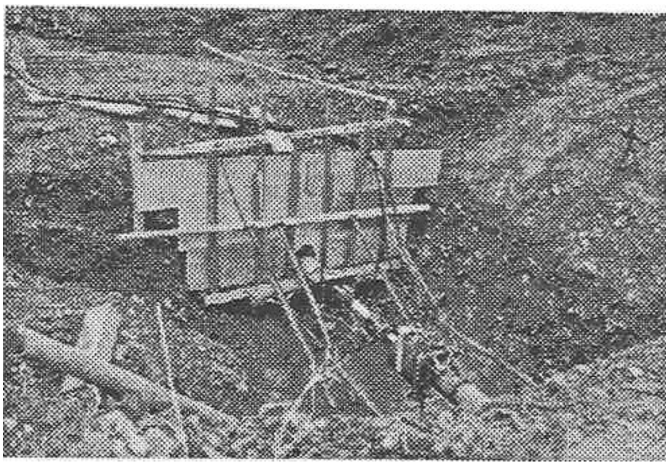
4.2 Pipeline Subjected to Axial Movement

4.2.1 Large Scale Experiment

The SPH pipe-soil intercation model was validated based on published lab and field full-scale experimental data:

1. **<u>University of Britsh Columbia (UBC):</u>** [6] performed large-scale tank experiments to investigate pipeline behaviors when subjected to axial soil movements. The test program consisted of four axial tests; the pipe loading tests were performed in a large soil chamber of 3.8 to 5.0m (length) x 2.50m (width) x 2.5m (height). A perspective view of the soil chamber (box) used for the axial pullout is shown in Figure 4(a). The UBC pipe had an outside diameter of 457 mm (18 in) and a wall thickness of 12.7mm (0.5in). The pipe was buried at an overburden ratio (H/D) of 2.5in a sand with friction angle of 36^0 and cohesion, c=0 kPa.
2. **<u>University of Italy:</u>** [7, 8] performed a series of four in situ full-scale pullout tests to investigate the behaviours of pipeline subjected to axial soil movements. These tests were conducted at two sites with out-of-service pipelines, located in northern and central Italy (Figure 4 b). The 24 inch diameter polyethylene wrapped pipe was buried at the overburden ratio (H/D) of 3.2in a clay with friction angle of 30^0 and average cohesion, c=38kPa.

(a) UBC Experimental Trial.



(b) Italian Full-Scale Pullout Test

Fig.4: Pipe Axial Displacement Experimental Trials

4.2.2 Finite Element Model

A 3D analysis using the LS-DYNA SPH technique was performed to simulate UBC and Italian trials. The FE model used for the analysis is shown in Figure 5, with the same geometry as the UBC experimental setup. The soil mass in the tank is taken to be stress free and have uniform properties in its initial state. The axial pipe displacement is imposed in two steps as described in Section 4.1.2.

4.2.3 Comparison of Analytical and Experimental Results

Figure 6 presents comparison of numerical and experimental force resistance versus displacement responses for the UBC tests conducted on loose sand (AB-5). A peak load of 9.7kN/m was reached in the experiment after a displacement of 10mm and this value dropped to a constant value of 8.5kN/m after an axial displacement of 150mm. The numerical model peak and post peak force redictions were 8.5kN/m and 8.1kN/m, respectively. These results are in good agreement with the measured forces.

Figure 7 presents a comparison of numerical and experimental force versus displacement response for the Italian trial conducted on natural clayed soil. A peak load of 30.2kN/m was reached in the experiment at a displacement of 3.6mm and this value drops to a constant value of 18kN/m after the axial displacement of 100mm. The peak and post peak forces predicted by the numerical model were 31kN/m and 21kN/m, respectively. These results are in good agreement with the measured forces.

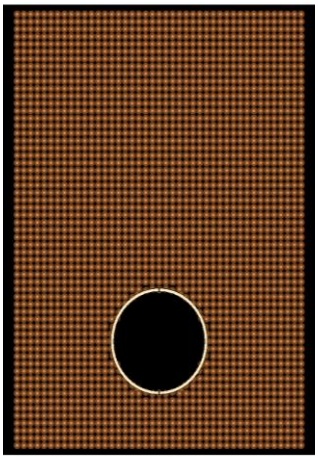


Fig.5: Illustration of the FE Model Including the Pipe.

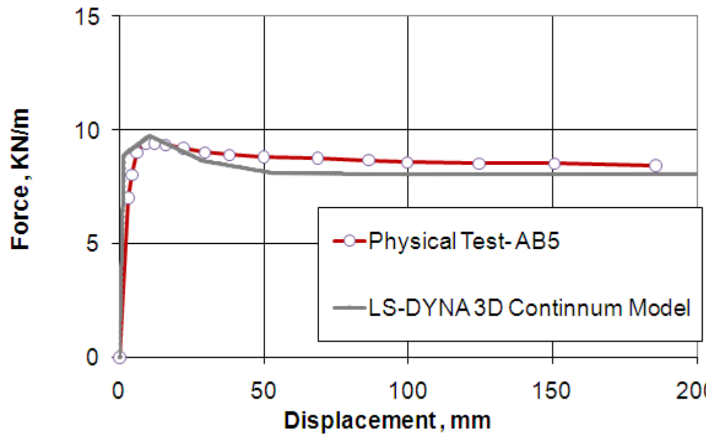


Fig.6: Comparison of Numerical Model and Test Results (UBC) for Loose Sand.

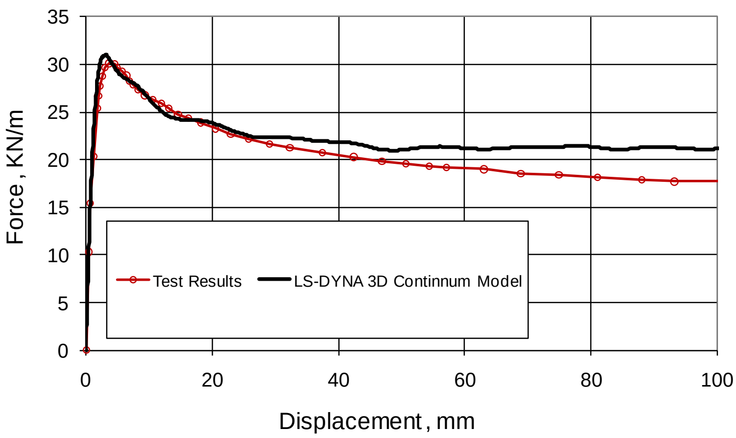


Fig.7: Comparison of Numerical Model and Test Results (Italy) for Clay Soil.

4.3 Pipeline Subjected to Surface Faulting

4.3.1 Large Scale Experimental Model

Large full-scale permanent ground deformations trials with high density polyethylene (HDPE) pipes were conducted by Cornell University [9] and used to validate the BMT SPH pipe-soil interaction model. The 407mm (16in) diameter pipe loading tests were performed in two movable 6.6m long basins, 3.2m wide by 2.3m deep. The experimental basins including 90 metric tons of partially saturated sand was displaced 1.22m relative to each other at a crossing angle of 65^0 as illustrated in Figure 8. The pipeline was installed with 0.9 m depth of soil over the pipe. The partially saturated sand (4% water content) had a dry unit weight of 15.5KN/m^3 and an average friction angle of 39.5 degrees.

4.3.2 Finite Element Model

An example of the SPH model used for the analysis is shown in Figure 9 with the geometry exactly similar to the experimental setup (Figure 8). The distance between the bottom boundary and the pipe is the same for both numerical and mesh and the tank experiment.

4.3.3 Analytical and Experimental Results

Figure 8 shows the test compartment [9] and the BMT validation model is presented in Figure 9. The figure illustrates the response of pipeline subjected to ground movement similar to the laboratory shear box test. Figure 10 compares the finite element model and experimental trial deformed pipe shape, illustrating that the SPH model was able to closely simulate the pipe deformation process.

Fig.8: Large-Scale Split-Box

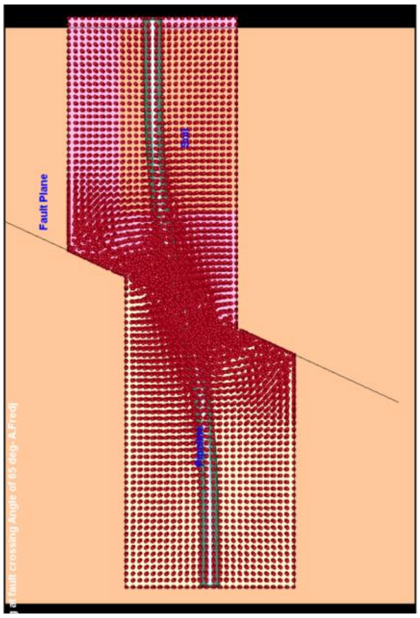


Fig.9: Illustration of The FE Model

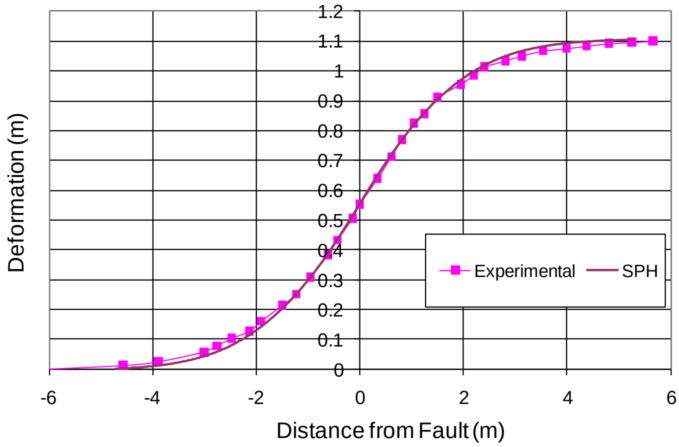


Fig.10: Pipe Deformation: Comparison of Numerical Model and Test Results

5 Pipeline Response to Slope Movement and Evaluation Strain Demand

5.1 Introduction

A major slope in southern Manitoba (Figure 11) has been experiencing deep seated movement of approximately 60mm per year. This 24m high x 85m long slope contains a right of way with five large diameter crude oil pipelines constructed from 1950 to 1998. It is estimated that the slope has moved up to 3m since pipeline installation. Management of the effect of this slope movement on the pipelines has involved cross-functional strategies that include geotechnical, integrity, and stress evaluations.

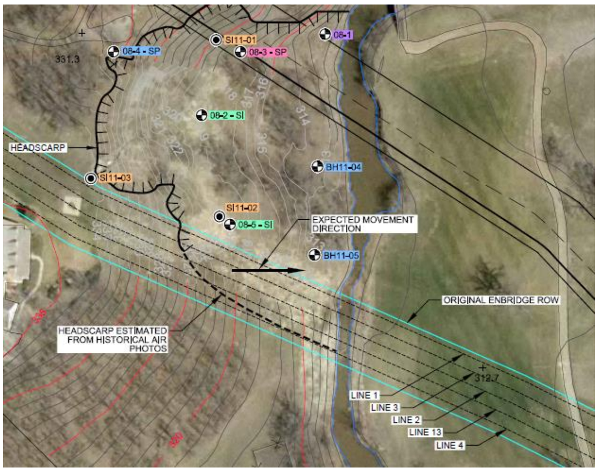


Fig.11: Plan View of Pipes and Slope

Line	OD [mm]	wt [mm]	MOP [MPa]	Grade	SMYS [MPa]	UTS [MPa]
1	508	11.9	10.0	X52	359	455
2	660	7.14	6.7			
3	864	7.14	3.6			
	864	9.53	3.6			
13	457	7.94	7.1	X46	317	434

Table 1: Pipe Input Parameters Used for the Analyses.

A finite element analysis (FEA), using SPH pipe-soil interaction model was completed to evaluate the pipeline stresses and strains caused by the slope movement to-date. The results indicated that strain capacity on one of the pipelines may be near its limit.

5.2 Numerical Simulation of Slope Movement

An SPH pipe-soil interaction finite element model was generated to evaluate the pipeline stresses and strains caused by the slope movement. The soil material is treated as particles having their masses smoothed in space using the SPH method. The pipe is modeled using Belyschko-Tasy shell elements with 5 integration point through thickness using material 24, MAT_PIECEWISE_LINEAR_PLASTICITY. Table 1 lists the linepipe properties considered. The full stress-strain curves for the pipe materials were developed based upon the CSA Z245 [11] minimum specified properties.

The soil was modeled using a double hardening plasticity model, material 5, MAT_SOIL_AND_FOAM. Two soil conditions were considered in this model, an undrained condition representing a sudden movement and a drained condition representing a gradual ground movement. The effects of the excess pore water pressures were not considered in this modeling exercise. The simulations were also limited to a single phase material response. The range of soil material model properties used in the continuum model is summarized in Table 2.

Soil Property (colluvium and clay fill)	Value(s)
Cohesion, C_u' drained (undrained) [kPa]	5 (45)
Friction angle, Φ_u' drained (undrained) [deg]	17 ,23, 25 (0)
Interface friction angle drained (undrained) [deg]	17 , 25 (35)
Soil saturated unit weight [kN/m^3]	19
Shear modulus drained (undrained) [MPa]	10 (17)
Average depth of cover to top of pipe [m]	1.5 to 1.8

Table 2: Soil Properties Used in the Continuum Model.

The ground profile, slip surface failure and pipeline profile were developed by considering field survey data, aerial photographs of the observed head scarp, a conservatively assumed toe location at the slope bottom, observed slide depths and directions from the in-place slope inclinometers, and in-line inspection position data for each of the five pipelines in the right of way. From this data several slope movement scenarios, parallel to the pipeline were developed, as shown in Figure 12. The slope toe low slip plane to pipe model angle, as shown on the right-hand side of Figure 12b), produced pipe and ground movements that best approximated field observations.

5.3 Finite Element Results

Figure 13 illustrates the pipe axial strain distribution developed by the numerical model for line 2 after 4.48m of slope movement. The maximum strains at the 6 O'clock and 12 O'clock positions are plotted illustrating the locations of greatest curvature and the inset exaggerated plot of the original and displaced pipe position can illustrate the displaced pipe shape. While the primary mode of pipe deformation is bending, in this case, the tensile and compression strains are not equal because there is a uniform axial strain promoting elongation of the pipe.

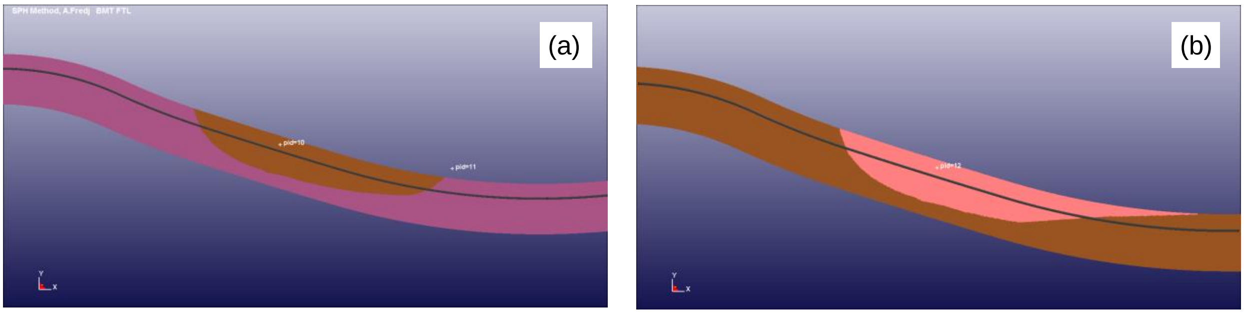


Fig.12: Illustration of the SPH Model – Slip Failure Surface.

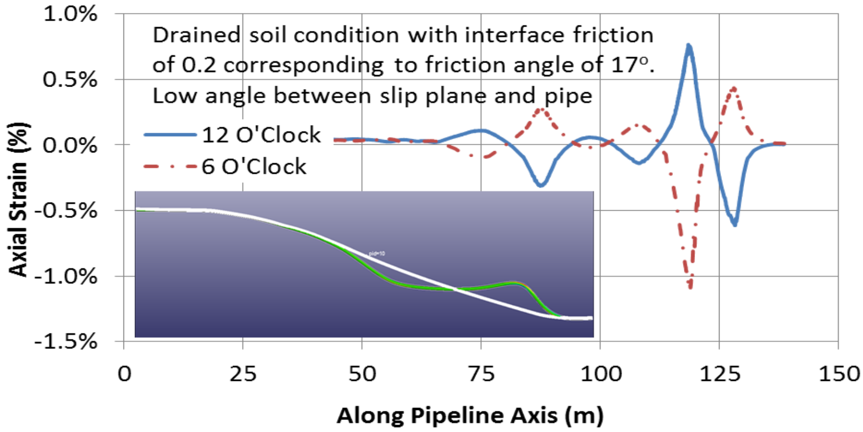


Fig.13: Illustration of the SPH Model – Slip Failure Surface.

5.3.1 Pipe Limit Strains

It was assessed that the slope movements generate axial strains in the pipe which are generally compressive at the toe of the slope and tensile at the top of the slope. Compressive strains may results in local buckling/wrinkling of the pipe. In this investigation the strains resulting in the onset of local buckling were assessed as defined in CSA-Z662 [11] strain limit design, API RP1111-Limit State Design [12], the University of Alberta [13] buckling limits and those developed from detailed finite element analysis [14].

Tensile strains acting on a significant weld flaw could potentially cause a girth weld failure. The maximum allowable tensile strains are governed by the failure stress or strain state for a circumferential defect. This assessment was completed using a British Standard 7190 Level 2A [15].

The response of the pipe-soil interaction model compared with the pipe limit strains as shown in Figures 14 and 15. The figures present the compression and tension limiting strains, where the tension strains consider 5mm long crack like defects of various depths. Two slope movement scenarios are presented in each figure considering drained and undrained soil conditions (gradual or sudden ground movements) for the low and high slip plane to pipe angles. The differences in strains accumulated in these scenarios provide bounds upon which to make integrity management decisions. The total estimated slope movement at the time of the analysis is represented by the heavy vertical line illustrating that the pipeline likely exceeded the permissible strains.

It is shown in Figures 14 to 16 that it was evaluated that the slope movements caused the compressive strain limit of Line 2 to be exceeded. This demonstrated that potential for buckle or wrinkle formation at select locations along the slope. An increase in the magnitude of the soil movement resulted in local buckling/wrinkle of the pipe in the compressive zone as shown in Figure 15. This is considered to be a serviceability limit state, and review of caliper inspection data did not reveal any geometric anomalies at this location. It was also assessed that the tensile strain capacity may have become exceeded, if there were large flaws present in the precise locations of high strain. While this was not expected, it was decided that GW inspections would be warranted.

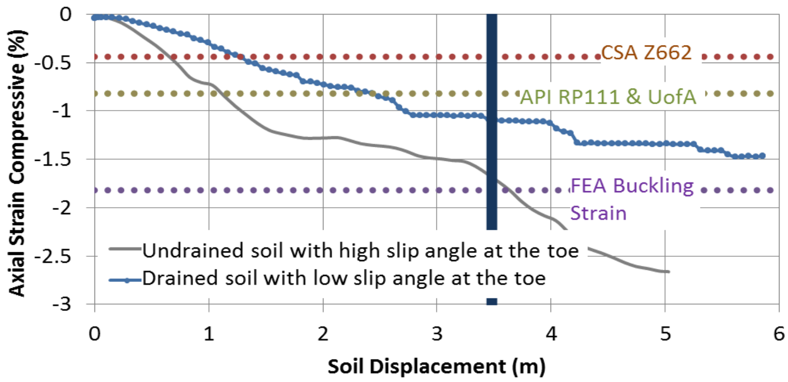


Fig.14: Illustration of the SPH Model – Slip Failure Surface.

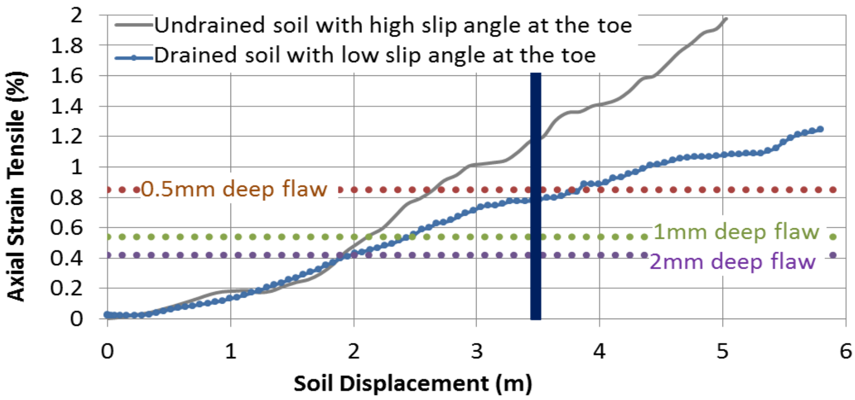


Fig.15: Illustration of the SPH Model – Slip Failure Surface.

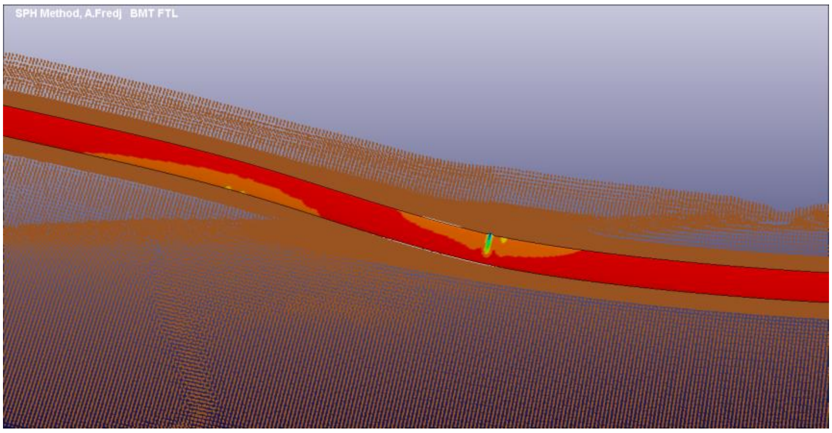


Fig.16: Illustration of the SPH Model – Slip Failure Surface.

5.4 Strain Relief

The results indicated that strain capacity on one of the pipelines may be near its limit. Correspondingly and in order to be conservative, a stress relief was conducted on three of the pipelines within the right-of-way. This mitigation involved excavating the pipelines 360 degrees with allowed for their decoupling from surrounding soil, and the associated pipeline spring back was surveyed. Figure 17 illustrate the strain relief program that was completed.

The strains from the pipe-soil interaction analyses were compared to those measured by a single run in-line inspection (ILI) inertial measurement unit (IMU). Both the ILI IMU data and the stress analysis result were used to consider the location of strain gauges that would be installed on the pipeline to record the effectiveness of the strain relief program and allow for future pipe monitoring.

Fig.17: Illustration of the Pipeline Strain Relief Program.

The numerical simulation results were compared with field survey measurements and strain gauge data. Figure 18 shows the measured pipeline elevation profile and local vertical spring back after excavation developed based upon field surveys that were conducted daily at the stress relief site. This data is compared to the FE model predictions developed prior to the excavation program.

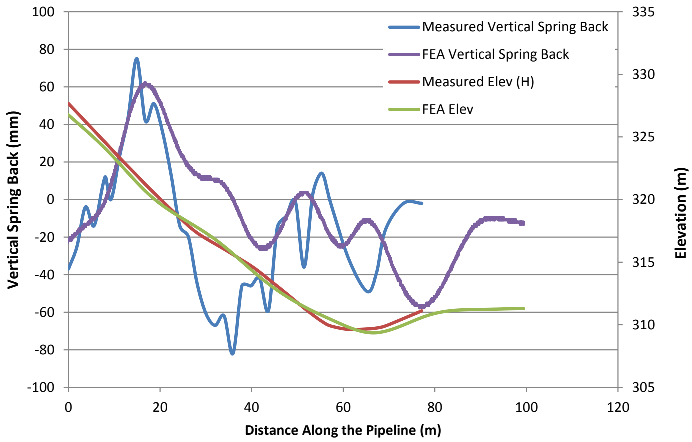


Fig.18: Comparison of Measured and Predicted Pipe Position and Spring –Back After Excavation.

Comparison of the finite element results and measured pipeline profile and spring back in general show good agreement, with the following general comments.

- There is good agreement between the measured and calculated pipeline profile. The agreement would be improved if the position along the pipeline of the measured pipeline elevations were off set a meter towards the bottom of the slope.
- Good agreement between the maximum measured and calculated pipeline spring back in the vertical and axial direction are observed. The maximum measured spring back is about 75mm and 40mm, and the calculated spring back is approximately 60mm and 35 mm, in the vertical and axial directions, respectively.
- The finite element analysis reported lateral pipeline spring back of 40mm versus the field measured 130mm of spring back. This result supports the pre-excavation program geotechnical observation that the pipeline was subjected to a lateral soil movement. This lateral soil movement was not explicitly considered in the finite element model and as such would not be expected to agree with the field observations.

A review of the strain gauge measurements was completed and good agreement was generally found. For example, a detailed review of the pipeline strain gauge data, for Station 4 on Line 2, resulted in observations such as those presented in Table 3.

Source	Strain Relief after Excavation	After Line Lowering
BMT FE Model	0.1 to 0.13%	0.2 to 0.25%
Strain Gauge	0.125%	0.2%

Table 3: Measured and Predicted Changes in Pipe Strain.

The strain gauge data indicated that the excavation produced local bending in the pipe (e.g. the change at the 12 O'clock position is not the same as the 6 O'clock position) which reversed some of the bending developed by the slope movement. The strain gauges at the 3 O'clock and 9 O'clock positions are similar in value so that they suggest no lateral bending, as planned.

6 Summary

This paper has described the application of a 3D continuum modeling technique for assessing the performance of a pipeline system subjected to large soil displacements. This analytic process made use of LS-DYNA SPH modeling capability, and can consider a wide range of soil types and soil movement scenarios. The results are compared with published experimental data of large-scale tests to verify the numerical analysis method. Comparison between numerical predictions and full-scale experiment results showed a good agreement. The SPH method as applied to the pipe soil interaction scenarios investigated has been demonstrated to accurately simulate the observed phenomena in terms pipeline response and soil deformation.

The validated SPH model was used to characterize a slope movement geotechnical hazard and its rate of progress on the accumulation of strains in the pipeline. These strains were compared to both finite element based failure criteria and those incorporated in pipeline design or integrity management standards. The models were useful in evaluating a range of scenarios to define operational safety and thus support maintenance action decision making. Field measurement result from the remedial action program demonstrated the agreement of the numerical model with the pipeline system response.

7 Acknowledgements

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