Integrated Framework in Support of Pipeline Engineering Design for Geohzards

Kenny, S.,
Department of Civil and Environmental Engineering – Carleton University, Ottawa, ON, Canada
Debnath, P., Pike, K., & Hawlader, B.
Department of Civil Engineering – Memorial University, St. John's, NL, Canada

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ABSTRACT
Energy pipelines are critical elements of the national infrastructure for the transportation of oil and gas resource. These pipeline systems may extend hundreds of kilometers in length, traverse across terrain units with varied geotechnical properties and may be impacted by geohazards. The relative ground movement imposes forces on the buried pipeline that may cause local damage and impair the mechanical performance with respect to serviceability or strength limits. The current state-of-practice for the engineering design and integrity assessment of a buried energy pipeline is based on structural pipe/soil interaction models idealized using beam and spring elements. This approach can be deficient when analyzing complex pipeline/soil interaction events that need to account for complex boundary conditions, load transfer processes and failure mechanisms. Continuum finite element methods can address these deficiencies but require an integrated framework including experienced numerical analysts, laboratory tests to define input parameters for soil constitutive models and physical data to verify simulation procedures. In this paper, the framework for integration of these technical approaches in support of pipeline engineering design is discussed with reference to recent studies. The potential for improving current pipeline engineering practice is also explored.

RÉSUMÉ
Les pipelines sont des éléments essentiels de l'infrastructure nationale pour le transport des ressources de pétrole et de gaz. Ces réseaux de canalisation peuvent s'étendre sur des centaines de kilomètres de long, traversant différentes formations géologiques avec des propriétés géotechniques variées, et peuvent être affectées par des risques géologiques. Des mouvements au sein du sol peuvent impliquer des forces sur le pipeline enfoui et causer des dommages locaux et ainsi porter atteinte au rendement mécanique par rapport aux limites de maintenabilité ou de résistance. Les pratiques actuelles en ce qui attrait à la conception technique et à l'évaluation globale d'un pipeline souterrain sont basées sur des modèles idéaux d'interaction entre la structure du tuyau et les sols en utilisant des éléments de poutres et de ressorts. Cette approche peut être déficiente lors de l'analyse d'événements d'intéractions complexes entre le pipeline et le sol qui doit tenir compte des conditions aux limites complexes, des processus de transfert de charge et des mécanismes de défaillance. Des méthodes d'éléments finis continus peuvent remédier à ces lacunes, mais nécessitent un cadre intégré, y compris des analyses numériques, des tests de laboratoire expérimentés afin de définir les paramètres d'entrée pour les modèles de constitution des sols et des données physiques pour vérifier les procédures de simulation. Dans cet article, le cadre d'intégration de ces approches techniques est discuté en référence à des études récentes. L'amélioration potentielle de la pratique est aussi explorée, quant à la conception technique des pipelines.

1 INTRODUCTION

Energy pipelines are critical elements of the national infrastructure used to transport oil and gas products that underpin the competitiveness and long-term growth of the Canadian economy, and the quality of life enjoyed by society. In 2013, the energy pipeline industry contributed $113 billion, representing more than one-quarter of the goods producing value to the national economy (CEPA, 2015).

In Canada, the buried transmission pipeline network exceeds 830,000 km, with greater network lengths in Europe and the US (CEPA, 2015). These pipeline systems extend across terrain with distributed topographical, hydrological, and geotechnical characteristics with varied intensity of anthropogenic activity. These local features may trigger geohazards (e.g., relative ground movement events including subsidence, slope stability, frost heave) that pose loads on the buried pipeline. The load effects may result in local stress or deformation of the pipeline section that may cause damage (e.g., global or local buckling) or failure (e.g., through wall rupture) to occur.

Although the frequency of pipeline rupture incidents due to geohazards is relatively low, typically less than 10% of total incidents (e.g., EGI, 2015; Jeglic, 2004), the consequences can be significant in terms of environmental damage, injury or loss of life. For example, in 2004 a natural gas pipeline, located in Ghislenghien, Belgium, ruptured due to external interference that resulted in the loss of 24 lives, 150 people injured and more than €100 million in property damage. Similar to other industries (e.g., aerospace), ensuing failure events, questions are often raised by the pubic and industry on the adequacy of current engineering practice, standards, regulations and policies to promote pipeline integrity and safety.

The current state-of-practice for the analysis, design and integrity assessment of buried pipelines subject to geohazards is based on structural pipe/soil interaction models idealized using beam and spring elements (e.g., ALA, 2005). For small deformation loading events, governed by equations of equilibrium, where the pie response is primarily elastic and specific problems with large deformation (e.g., fault movement), there is sufficient guidance within a sufficiently acceptable technical framework.

This approach can be deficient for the analysis of other pipeline/soil interaction events that may involve complex route alignment (e.g., back-to-back bends), oblique load coupling and large deformation geohazards such slope movement or ice gouging (e.g., Daiyan et al., 2011; Nobahar et al., 2007; Roy et al., 2015). Continuum finite element methods can address these deficiencies but require an integrated framework including experienced numerical modellers, laboratory tests to define input parameters for soil constitutive models and physical data to verify simulation procedures (Pike et al., 2013,2014; Roy et al., 2014,2015).

As shown in Figure 1, fully coupled pipeline/soil interaction events involve complex interdependent relationships among the system demand, response and capacity. The system demand (i.e., hazards and loads) includes anthropogenic (e.g., mining, agriculture practices) and natural geohazards. System response directly addresses the geotechnical conditions (e.g., soil type, strength parameters) and pipeline characteristics (e.g., diameter, internal pressure, operating temperature) with respect to the load transfer, load effects and soil failure mechanisms. Performance limits on the pipeline mechanical response are addressed through the system capacity (e.g., stress, ovalization, local buckling, rupture) and parameters that may have statistically significant interactions.

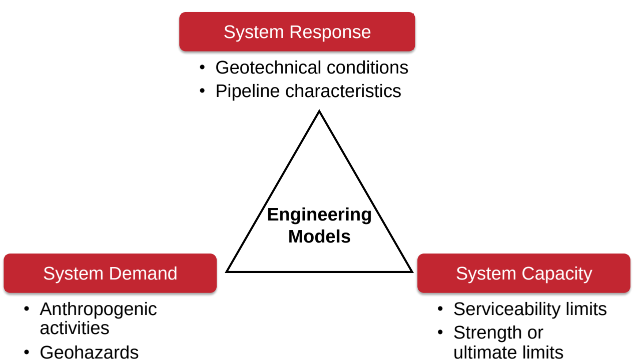

Figure 1. Major elements and parameters for coupled pipeline/soil interaction event

An integrated research framework is needed to establish confidence in practical, reliable, and cost-effective engineering tools that can be used support pipeline design and operations. As part of this framework, laboratory testing can be used to advance constitutive models, while physical modeling can be used to evaluate hypotheses. The results from these studies can be used to verify numerical simulation tools that can assess system demand, response and capacity across a range of practical design and operational conditions. The research outcomes can then be used to advance engineering practice, guidance documents, and governing codes and standards.

In this paper, this integrated research framework is addressed through examination of recent studies that have included laboratory testing, centrifuge and full-scale modelling, and numerical simulation to address pipeline/soil interaction events for energy pipelines. The framework in support of pipeline engineering design is discussed. The potential for improving current pipeline engineering practice is also explored.

2 CENTRIFUGE MODELLING STUDIES

A series of reduced scale centrifuge tests examining oblique pipeline/soil interaction (Daiyan, 2013; Daiyan et al., 2011; Debnath, 2015) and full-scale lateral pipeline/soil interaction tests (Burnett, 2015) have been recently conducted. The principal objective for these tests was to support the development and verification of continuum finite element modelling procedures.

A rigid pipeline was pulled through a prepared dry, cohesionless soil test bed, in the loose and dense conditions, to examine the load-displacement response, pipe/soil interaction response, soil failure mechanisms and strain localization. The instrumentation included load cells to measure soil forces, potentiometers, LVDT's and lasers to measure displacement and, for the full-scale tests, Digital Image Correlation (DIC) technique was used to measure soil deformations and strain localization.

Further discussion on the reduced-scale centrifuge test apparatus and procedures is presented in the following subsections.

2.1 Apparatus and Instrumentation

Centrifuge modelling provides a basis to simulate soil/structure interaction problems where the prototype soil stress field, due to the effects of gravitational acceleration, can be simulated at reduced scale through the laws of similitude (e.g., Craig, 1995; Garnier et al., 2007; Gaudin et al., 2016).

A series of tests were conducted using the C-CORE geotechnical centrifuge located on the campus of Memorial University (Daiyan, 2013; Debnath, 2015). As shown in Figure 2, a rigid pipeline [1] was pulled through a dry, cohesionless test bed to examine the coupled axial-lateral pipeline/soil interaction response for oblique loading events. The axial and lateral loads were measured through two bi-axial load cells [2,3], and the pipeline was pulled through the soil via a supporting structure consisting of two stanchions [4,5] that was braced by a dogbone [6].

As shown in Figure 3, each stanchion was connected to a ball race [7,8], which allowed for vertical motion. The ball races were connected to a guiding plate [9] that was connected to a motorized carriage [11] proving the means to move the pipe through the soil. The guiding plate was also used to orientate and fix the longitudinal axis of the buried pipeline relative to the direction of motion (e.g., pure axial, pure bearing, oblique motion) in the soil test bed. Two linear variable differential transformers (LVDT), with one shown as [10] in Figure 3, were used to measure the pipe vertical movement. Two lasers, with one shown as [12] in Figure 3, were used to measure the pipe lateral or horizontal displacement.

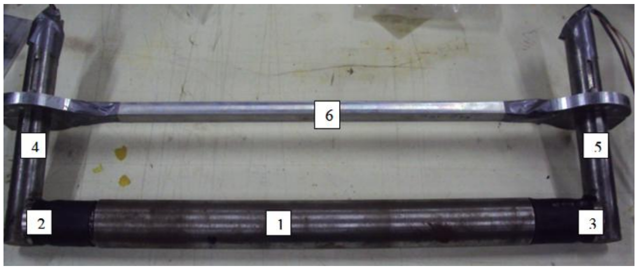

Figure 2. Pipe assembly [1] with biaxial load cells [2,3] stanchions [4,5] and dogbone [6] (Debnath, 2015)

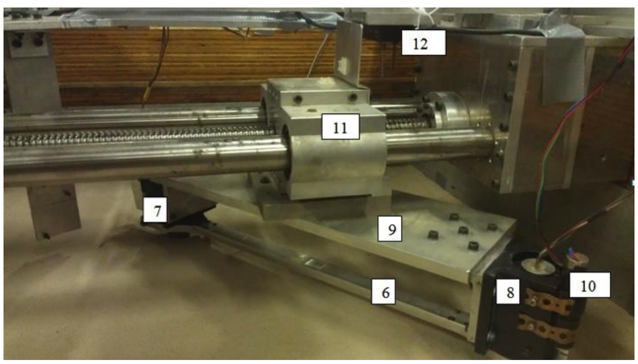

Figure 3. Major elements of the centrifuge test apparatus including the dogbone [6], ball race [7,8], guiding plate [9], LVDT [10], motorized carriage [11] and laser [12] (Debnath, 2015)

2.2 Pipe/Soil Interaction Test Parameters

The centrifuge test bed was a dry, cohesionless soil using dry fine silica sand in the loose (Debnath, 2015) and dense (Daiyan, 2013; Debnath, 2015) condition. A summary of the soil properties is presented in Table 1. The acceleration field for the tests conducted by Daiyan (2015) and Debnath (2013) was 12.3g and 13.25g, respectively. A summary of the test parameters for the centrifuge strong box and rigid pipeline are summarized in Table 2.

The friction angles were measured using triaxial cell (Daiyan, 2013) and direct shear box (Debnath, 2015) apparatus. Magnitudes of the peak and critical state friction angle for loose sand may be considered on the higher end of the range.

In terms of the relative angle of attack between the pipeline and soil, Daiyan (2013) conducted 4 experiments that examined axial (0°), lateral bearing (90°) and 2 oblique loading conditions (40° & 70°). In the study conducted by Debnath (2015), for the dense test bed condition there were 6 experiments conducted, which examined relative angle of attack between the pipeline and soil for axial (0°), lateral bearing (90°) and 4 oblique (20°, 40°, 50° & 70°) loading conditions. In the loose test bed condition, 4 tests were conducted that examined axial (0°), lateral bearing (90°) and 2 oblique (40° & 70°) loading conditions.

Table 1. Soil test bed parameters

Parameter	Value	
	Daiyan (2013)	Debnath (2015)
Average particle size, d_{50} (mm)	–	0.22 mm
Coefficient of uniformity (#)	–	1.92
Average dry density, loose (kg/m^3)	N/A	1467
Relative density, loose (%)	N/A	33
Peak friction angle, loose (°)	N/A	40
Constant volume friction angle, loose (°)	N/A	36
Average dry density, dense (kg/m^3)	1598	1567
Relative density, dense (%)	82	72
Peak friction angle, dense (°)	43	47
Constant volume friction angle, dense (°)	33	40

Table 2. Centrifuge strongbox and model pipeline parameters

Parameter	Value	
	Daiyan (2013)	Debnath (2015)
Acceleration field, g	12.3	13.25
Pipe diameter (mm)	41	46
Pipe/soil interface friction coefficient (#)	0.44	N/A
Centrifuge strong box (mm × mm × mm)	1180 × 940 × 400	
Pipe length to diameter ratio, L/D (#)	8	
Pipe burial depth at springline to diameter ratio, H/D (#)	2	

2.3 Axial Pipe/Soil Interaction

In this section, test data for axial (0°) pipeline/soil interaction (Daiyan, 2013; Debnath, 2015), obtained using the geotechnical centrifuge, were analysed and compared with public domain datasets from full-scale tests (Hsu et al., 2001, 2006; Karimian, 2006). The physical test data is presented in Figure 4 where the normalized axial force is presented as a function of the normalized axial displacement. The measured normalised axial force was also compared with guidance from other studies (Schaminee et al., 1990) and current engineering practice (ALA, 2005).

The normalized axial force was defined as the real axial force per unit length divided by the dry soil unit weight (γ), the burial depth to the pipe springline (H_s), and pipe diameter (D). The normalized axial yield displacement was defined by the mobilized soil displacement at the axial yield force divided by the pipe diameter (D). The ALA (2005) guidelines define the yield displacement as 3 mm and 5 mm for dense and loose sand, respectively. The study by Schaminee et al., (1990) did not define an axial yield displacement criterion.

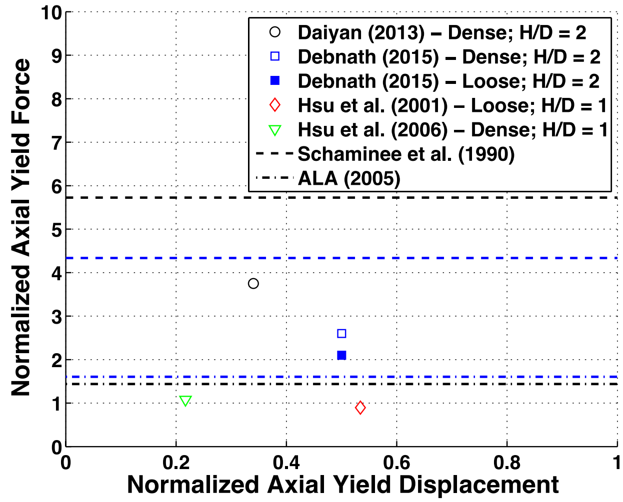

Figure 4. Normalized axial force and yield displacement for axial pipe/soil interaction

As shown in Figure 4, for an H/D of 2, the measured normalized axial yield force was greater than the ALA (2005) guidelines by a factor of 2.6 and 1.6 for the centrifuge tests conducted by Daiyan (2013) and Debnath (2015), respectively. In the full-scale tests, Hsu et al. (2006) observed normalized axial yield forces less than 1 for dense sand (peak friction angle of 42°) across the range of H/D (H/D = 1,2,3) examined, which was less than the ALA (2005) guidelines.

For cohesionless soil, the ALA (2005) guidelines define the axial force as

$$t_u = \pi \gamma H_s D \left(\frac{1+K_o}{2} \right) \tan \delta \qquad [1]$$

where γ is the effective unit weight, K_o is the coefficient of earth pressure at rest and δ pipe/soil interface friction angle. A coating dependent friction factor of 0.7 was used to represent smooth steel. In the study by Hsu et al. (2001), direct shear test results indicated a friction coefficient of 0.67 the soil friction angle. For dense sand, the K_o value was estimated using the expression presented by Sherif et al. (1984).

The experimental apparatus, instrumentation and procedure for the centrifuge (Daiyan, 2013; Debnath, 2015) and full-scale (Hsu et al., 2006) tests were similar. Differences in the test results may be attributed to the boundary conditions, pipe self-weight and local failure mechanisms.

Due to the pipe and soil placement processes, there may be local variation and gradient in soil properties (e.g., relative density) in comparison with the far-field conditions. This could influence soil mechanical behaviour (e.g., strength, dilation). The expectation would be a reduced peak friction angle and lower relative density that tends to decrease the yield axial force.

The centrifuge tests placed high-density, compliant foam blocks on the leading face of the stanchions, to promote local failure during the movement of the test frame through the soil, and thereby reduce end-bearing loads effects being transmitted onto the pipe assembly (Figure 2). The stanchions would impose bearing load on the foam to mitigate soil deformations and failure processes on the leading face. The tests conducted by Hsu et al. (2001, 2006) used a rigid blanking plate on the end pipe segment. The difference in compliance and boundary conditions may account for the discrepancy between the physical models.

To maintain rigid condition, the model pipelines used in the centrifuge studies were thick-walled pipe with a diameter-to-thickness (D/t) ratio of 6 (Daiyan, 2013) and 12 (Debnath, 2015). Conventional onshore energy pipelines would have a D/t ratio between 50 and 80, whereas offshore pipelines would have a typical D/t range of 30 to 45. The thicker wall increased the pipe self-weight, which increased the average local stress at the pipe springline. Accounting for this stress component, the axial resistance can be expressed as (Schaminee et al., 1990)

$$t_u = \frac{\pi}{4} \gamma H_c D \left(2 + 2K_o + \beta + K_o \frac{D}{H_c} \right) \tan \delta \qquad [2]$$

where H_c is the cover depth to the pipe crown, and β is the pipe weight normalized by $\gamma H_s D$. Accounting for the effect of the pipe self-weight, estimates of the soil axial force, using Equation [2], were greater than the centrifuge results by a factor of 1.5 and 1.7 for the tests conducted by Daiyan (2013) and Debnath (2015), respectively.

In a recent study conducted by Karimian (2006), for well-defined and controlled axial pull-out tests, the axial soil resistance for loose sand test bed conditions was consistent with estimates using the ALA (2005) guidelines, as defined in Equation [1]. For dense sand conditions, however, Karimian (2006) concluded the measured peak soil forces were 2 to 3 times greater than the ALA guidelines. The difference was attributed to increased resistance due to localized failure during shear deformation associated with constrained dilation around the pipe circumference. The shear zone was focused across a small annular layer with a thickness of 1 m to 3 mm or approximately 10 d_{50}. An equivalent lateral earth pressure coefficient was defined that was a function of the pipe diameter, soil elastic modulus and internal friction angle.

For the centrifuge tests parameters, the equivalent lateral earth pressure coefficient as defined by Karimian (2006) would range from approximately 1.0 to 2.25. Using Equation [1] with equivalent lateral earth pressure coefficients of 2.6 and 1.6 would match the centrifuge tests conducted by Daiyan (2013) and Debnath (2015), respectively. These observations would hold true for the tests in loose sand conducted by Debnath (2015).

In the tests conducted by Debnath (2015), the peak axial force for dense sand was 1.2 times greater than the loose sand test bed condition. In large scale axial pipe/soil interaction tests, Paulin et al. (1998) found that post-peak axial loads in dense sand were on average approximately 1.6 times greater than tests on loose sand.

The mobilization of yield displacement for the physical tests, ranging from 0.2D to 0.5D, were significantly greater than the estimates using ALA (2005) guidelines and results of Karimian (2006), which are on the order of millimetres (i.e., 0.005 D to 0.01 D). This may be due to the test setup, end bearing effects and boundary conditions for the centrifuge (Daiyan, 2013; Debnath, 2015) and full-scale (Hsu et al., 2001,2006) studies.

In addition, as presented by Daiyan et al., (2011) through numerical simulation, the axial load-displacement response becomes moderated at low oblique attack angles, which can be associated with misalignment of the test frame, where the peak load occurs at greater mobilization distances. In a physical test, the existence of any bifurcation, at low mobilization distances (e.g., < 0.1D), may not be discernable and thus account for the higher peak forces and mobilization distances to yield being reported.

2.4 Oblique Loading

Early studies on the effects of oblique pipeline/soil interaction events were based on analytical solutions (e.g., limit load analysis) that were complemented by analogue events such as inclined plate anchor tests (Meyerhof and Hanna 1978; Nyman, 1984). More recently, an increasing number of studies have investigated the load coupling effects during oblique pipeline/soil interaction events (e.g., Daiyan, 2013).

Based on the centrifuge tests conducted by Daiyan (2013) and Debnath (2015), as shown in Figure 5, the normalized axial (0°) and lateral (90°) interaction forces are coupled and dependent on the pipeline/soil attack angle. For these centrifuge tests, the coupled interaction was associated with two mechanisms characterized by the bounding surfaces of two yield envelopes. At low interaction angles (i.e., oblique attack angles) the soil failure was controlled by shear strength and friction along the pipeline/soil interface. This failure mechanism was associated with increased effective axial force with increasing attack angle. This has been observed in numerical studies on oblique pipeline/soil interaction in cohesive (Phillips et al., 2004a; Pike and Kenny, 2012a,b) and non-cohesive soil conditions (Daiyan, 2013; Daiyan et al., 2011,2010). The axial loads can increase by a factor as high as 2.5 for oblique attack angles less than 40°. The continuum finite element models on dense sand by Dayian (2013) was in good agreement with the centrifuge data with the greatest discrepancy between the physical and numerical model observed for the axial loading condition.

As the attack angle increases, the axial-lateral load coupling effects are moderated where the failure mechanism is governed by shear failure through the soil mass. For the lateral (90°) normalized bearing force, the centrifuge data is consistent with some public domain data (e.g., Audibert and Nyman, 1977; Calvetti et al., 2004) but is greater than current practice (e.g., ALA, 2005) by a factor of 1.35 and 1.6 for loose and dense sand, respectively. The full-scale test by Hsu (2001,2006) underpredicts the normalized bearing load by a factor of 0.57 and 0.81 for loose and dense sand, respectively.

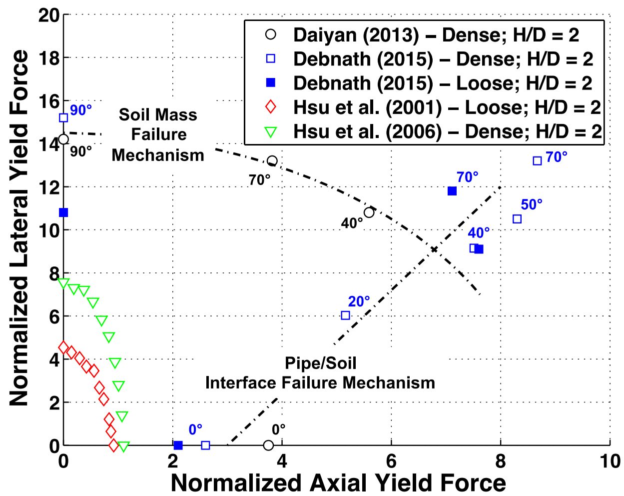

Figure 5. Normalized lateral-axial interaction

Through a parameter analysis, Daiyan (2013) has shown a family of yield envelopes (as shown by the dash-dot lines in Figure 5) exist that is a function of the soil peak friction angle, pipe/soil interface friction angle, and burial depth (H_s / D). Using Design of Experiments (DoE) techniques, empirical relationships defining the yield envelope were established. There was agreement between the numerical simulations and centrifuge data.

The greatest uncertainty lies with the differences between the geotechnical centrifuge and full-scale physical model results, particularly for the oblique loading cases. The centrifuge tests (Daiyan, 2013; Debnath, 2015) exhibited greater coupling between axial-lateral forces with attack angle than the full-scale tests (Hsu et al., 2001,2006). This degree of coupling is supported by results from continuum finite element simulations (e.g., Daiyan, 2013; Daiyan et al., 2011; Phillips et al., 2004a; Pike and Kenny, 2012a,b). There may be two reasons for this discrepancy between the physical modelling approaches related to the boundary condition and geotechnical centrifuge scaling laws.

As discussed in the Section 2.3, although the physical models had similar test conditions and procedures, one of the differences was treatment of the end bearing boundary condition. Differences in end compliance may influence the bearing loads and soil failure mechanisms on the leading face of the pipe during the interaction event that may influence the measured axial load response and be a moderating factor that accounts for differences between the two datasets.

A study conducted by Palmer et al. (2003) on the upheaval buckling response of pipelines observed the mobilization distance did not scale, between reduced-scale centrifuge and full-scale physical modelling tests, with respect to soil particle size. The comparative uplift resistance, however, was consistent. The mobilization error was related to soil kinematics and failure mechanisms associated with strain localization and formation of shear zones. The centrifuge mobilization distance was a fraction of the pipe diameter, which is similar to the observations in this study, and moderated the characteristic shape of the load-displacement relationship. This effect was related to the relative displacement and the evolution of shear stress.

For the oblique pipe/soil interaction tests, these scaling effects may, in part, influence the mobilization distance (Figure 4) and relative displacement that affects soil kinematics and formation of shear zones. This would, in turn, influence the determination of the peak loads that may influence the shape of the coupled yield envelope (dash-dot lines) as shown in Figure 5. For shallow buried pipelines, the lateral interaction factor is governed by a passive failure wedge and dependent on the soil weight (e.g., Daiyan, 2013; Phillips et al, 2004b; Rossiter and Kenny, 2012).

The potential influence of centrifuge scale effects, with respect to the vertical mobilization distance as observed by Palmer et al. (2003), on the yield load and displacement to peak load for lateral pipe/soil interaction events is shown in Figure 6. The centrifuge test results exhibit higher normalized mobilization distances to yield force that may be related to kinematics, soil failure mechanisms, and formation of shear zones. This would moderate the load-displacement response and interpretation of the peak load as shown by Method 1 and Method 2 in Figure 6. The peak load using Method 1 was based on the intersection of two tangents to the load-displacement relationship as proposed by Wantland et al. (1982). Using Method 2, based on the work of Terzaghi's, defines the peak load at the point of tangency for the nonlinear plastic deformation. There exists confidence in the centrifuge results, however, where the finite element simulations conducted by Daiyan (2013) are consistent with the physical test data. Other factors that may influence this response include the soil particle shape, gradation, and relative density. This issue requires further detailed investigation.

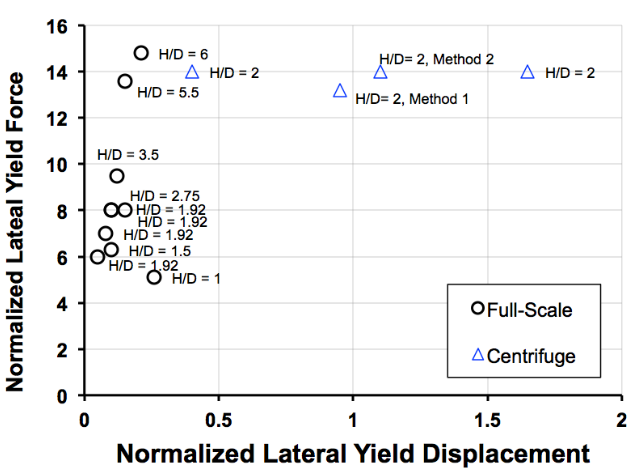

Figure 6. Normalized lateral-axial interaction

3 ADVANCING PRACTICE AND STANDARDS

3.1 Overview

Current engineering practice (e.g., ALA, 2005) recommends the use of numerical modelling procedures employing decoupled, structural beam/spring (i.e., structural) system for the analysis of pipeline/soil interaction events. This methodology provides a relatively simple, cost-effective strategy to support pipeline engineering design across a range of practical considerations (e.g., conventional stress based problem, above ground transitions).

There is a need to advance these computational models in support of pipeline design and integrity management for more complex scenarios. As the energy industry ventures into frontier regions, the operational requirements (e.g., high pressure, temperature) and environmental loads (e.g., frost heave, ice gouging) are becoming more challenging that imposes demands on the development of effective engineering solutions.

3.2 Enhanced Structural Models

For stress-based design, pipeline/soil interaction events subject to multi-directional loading (e.g., upheaval buckling, ice gouging, slope failure) requires an enhanced structural model (e.g., Kettle, 1984) due to the coupled load response and failure envelope (e.g., Figure 5).

Improved numerical tools to address this coupled interaction include macro-elements (e.g., Cochetti et al., 2009a,b) and finite element structural modelling procedures (e.g., Daiyan, 2013). These enhanced structural models account for the effects of mechanical load and displacement coupling. The soil yield envelope is defined in terms of the peak load and yield displacement along three orthogonal planes for specific design parameters (e.g., pipe diameter, pipe burial depth, soil type and strength properties and attack angle).

Physical models, which may be integrated with numerical simulations to extend the parameter database, provide the basis to define the nonlinear soil response due to soil deformations, material behaviour and load coupling.

3.3 Continuum Computational Models

For complex pipeline/soil interaction events, there is a need for more complex, computational modelling procedures (e.g., continuum finite element methods) to address strength evolution with deformation (e.g., change in peak stress, residual stress, dilation due to strain softening), load and stress path dependency, and deformation mechanisms (e.g., strain localization, shear bands).

The primary constraints on using these tools are requisite skills and expertise of the analyst, laboratory test data used to refine constitutive models, physical test data to verify the computational modelling procedures, and software and hardware requirements to conduct the simulation. Recent studies, using verified continuum finite element modelling procedures (e.g., Daiyan, 2013; Pike et al., 2012a,b,2013,2014; Rossiter and Kenny, 2012; Roy et al., 2014,2015) have supported these observations but have also shown the future potential of these tools to address complex problems, support engineering design and operations, and enhance engineering standards (e.g., Phillips et al., 204a,b; Pike et al., 2014).

4 CONCLUSIONS

Physical modelling and numerical simulations have demonstrated the significance of load coupling effects during oblique pipeline/soil interaction events. There is observed agreement between reduced scale-centrifuge and continuum finite element modelling procedures on the load coupling effects for lateral-axial pipeline/soil interaction in cohesive and cohesionless soil. There exists uncertainty on the characteristics of the yield envelope for oblique lateral-axial interaction events when these observations are compared with available full-scale data. This may be related to differences in the end boundary conditions used in the physical models and potential scaling errors in the centrifuge tests with respect to mobilization distance to yield. Future tests should conduct direct comparison (i.e., equivalent pipe and soil parameters) at reduced-scale and full-scale to resolve this uncertainty. Although there are some constraints, the use of continuum finite element modelling procedures will become more common place in supporting engineering design and can be used to advanced current engineering practice.

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