Effects of Geohazards on Energy Pipelines: Technology Framework Supporting Engineering Design and Emerging Tools

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ABSTRACT

Energy pipeline transportation systems are typically buried and may be subject to large deformation ground movements associated with operational parameters and natural events. Although modern computational tools provide robust, effective and efficient platforms for simulation of pipeline/soil interaction events, a comprehensive technology framework is needed to advance technical solutions in support of engineering design with confidence. This technology framework requires laboratory tests to refine constitutive models used within numerical algorithms and physical models to calibrate and evaluate the computational simulations. In this paper, elements of this technology framework are examined and results discussed from a research program developing advanced computational simulation tools, to assess the effects of large deformation ground movement events on buried pipelines. The importance of mesh topology, large deformations and strains, nonlinear material behaviour, contact mechanics and bifurcations on the strategies employed to develop the computational tools are explored. A discussion on current and future trends is also presented.

RÉSUMÉ

Systèmes de transport par pipeline de l'énergie sont généralement enterrés et peuvent être soumis à d'importants mouvements de sol de déformation associés à des paramètres de fonctionnement et les événements naturels. Bien que les outils informatiques modernes offrent des plates-formes robustes et efficaces pour la simulation d'événements d'interaction pipeline/sol, un cadre technologique globale est nécessaire pour promouvoir des solutions techniques à l'appui de la conception technique avec confiance. Ce cadre de la technologie exige des tests de laboratoire pour améliorer les modèles de comportement utilisés dans les algorithmes numériques et des modèles physiques pour calibrer et évaluer les simulations informatiques. Dans le présent document, les éléments de ce cadre de la technologie sont examinés et discutés les résultats d'un programme de recherche de développer des outils de simulation informatiques avancées, afin d'évaluer les effets des grands événements déformation de mouvements de terrain sur les canalisations enterrées. L'importance de la topologie de maillage, de grandes déformations et les souches, comportement des matériaux non linéaire, mécanique du contact et bifurcations sur les stratégies utilisées pour développer les outils de calcul sont explorées. Une discussion sur les tendances actuelles et futures est également présenté.

1 INTRODUCTION

Energy pipeline transportation systems provide an efficient, safe and economic mode of transport for the delivery of hydrocarbon products from resource areas to industrial and civilian end users. In 2012, the Canadian Energy Pipeline Association estimated the pipeline industry contributed more than \$8.8 billion to the Canadian Gross Domestic Product (GDP) and more than \$84 billion of hydrocarbon products for export. There is an estimated 830,000 km network of gathering, transmission and distribution pipelines servicing the oil and gas energy industry (CEPA, 2014). The estimated failure frequency for energy pipelines, measured as an annual frequency per 100 km pipeline system, was 10^{-3} to 10^{-2} for North America and Europe for data records back to 1980 (ASME, 2008).

The vast majority of onshore and offshore energy pipelines are in direct contact with the soil surface or are fully buried. The pipeline may be embedded within the soil to address requirements for flow assurance (e.g. thermal resistance), operational loads (e.g. upheaval buckling), external forces due to wave and current loads (e.g. on-bottom stability), and to afford protection from natural geohazards (e.g. slope failure, fault movement, ice gouging) and anthropogenic activities (e.g. fishing gear impact, excavator damage, mine subsidence). In this study, the effects of large deformation geohazards on buried energy pipelines are examined.

Buried pipelines are subject to operational loads (i.e. differential temperature and pressure relative to the ambient environment) and may be subject to soil loads or deformations due to geohazards (e.g. subsidence, seismic faulting, long-term slope movement). Current industry practice for simulating the pipeline-soil interaction event and estimating the pipeline`s mechanical response is primarily based on structural finite element modelling procedures (e.g. ALA, 2005; NEN, 1991). These industry recommended practices idealize the generalized pipe-soil continuum (Figure 1a) as a series of beam and spring elements (Figure 1b) that represent the mechanical response of the pipe and soil, respectively. The model is typically formulated using finite element methods (e.g. Kenny et al., 2000).

The pipe is modeled using beam theory with additional variables accounting for the effects of internal pressure and thermal expansion. The pipe's constitutive behaviour is generally defined as elastoplastic with yielding characterized by the von Mises criterion and isotropic hardening. The soil continuum response is idealized by a series of discrete springs connected to the pipe. The spring elements represent the soil force-displacement response per unit length of pipe, which act on three, mutually orthogonal axes to the pipe centreline as defined by the longitudinal, transverse horizontal, and transverse vertical directions. The soil spring force-displacement relationships may be defined as bilinear (i.e. elastic-perfectly plastic), piecewise multilinear, or hyperbolic functions to represent the nonlinear mechanical behaviour of the soil.

For simple loading conditions (e.g. response to operational loads, unidirectional loading) with force (i.e. natural) boundary conditions, or small amplitude, relative ground movement events, the idealized structural model provides reasonable estimates of the mechanical behaviour of the pipe and soil. For large deformation and large strain pipe-soil interaction events; such as fault movement, frost heave, ground subsidence, ice gouging, thaw settlement and upheaval buckling (e.g. Bruschi et al., 2010; C-FER, 1995; Einsfeld et al., 2003; Kenny et al., 2000,2007; Nixon et al., 1996; Xie et al., 2013), the idealized structural model has been used to support pipeline design. There exists uncertainty, however, regarding the technical basis and reliability of engineering outcomes for application of the structural beam/spring modelling approach to solve problems with large relative ground movement events (e.g. Konuk at el., 2006; Peek and Nobahar, 2012; Nobahar et al., 2007).

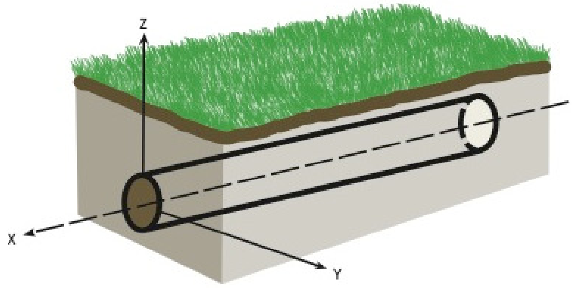
From a practical engineering perspective, there are many positive attributes for the structural beam/spring modelling approach to the simulation of pipe-soil interaction events that relate to the relative simplicity and utility of the formulation. The structural modelling procedures can be implemented within a numerical framework that imposes limited constraints on the proficiency and specialized skill set of the practising engineer, numerical platform with respect to the hardware and software requirements, while providing a technical basis to conduct extensive parameter studies within short time frames (e.g. Kenny et al., 2000, 2007).

For many large deformation pipeline-soil interaction problems, however, a complex relationship exists between the pipe and soil that may be associated with load transfer processes, pipeline deformations, soil failure mechanisms, contact mechanics, and strain localization (e.g. Peek and Nobahar, 2012; Pike and Kenny, 2012; Pike et al., 2012,2013; Jung and Zhang, 2011; Robert, 2010). Key parameters include the pipeline characteristics (e.g. diameter, burial depth, operational load conditions), and the soil's physical properties (e.g. water content, pore pressure) and strength characteristics (e.g. friction or dilation angle). The uncertainty associated with structural based models is primarily related to the idealization of a continuum response using discrete elements that are defined by generalized force-displacement relationships. For these events, the idealized beam/spring mathematical construct cannot provide an adequate characterization of more realistic and complex soil response with respect to strength behaviour and evolution with deformation (e.g. strain softening, compaction, dilation, path dependency), rate effects (e.g. pore pressure, consolidation) and load coupling (i.e. complex loading on more than one orthogonal soil spring axis). Several studies have demonstrated the effects of geometry, boundary and loading conditions, and relative stiffness on the resultant soil response (e.g. Konuk at el., 2006; Peek and Nobahar, 2012; Nobahar et al., 2007; Pike et al., 2011a,b).

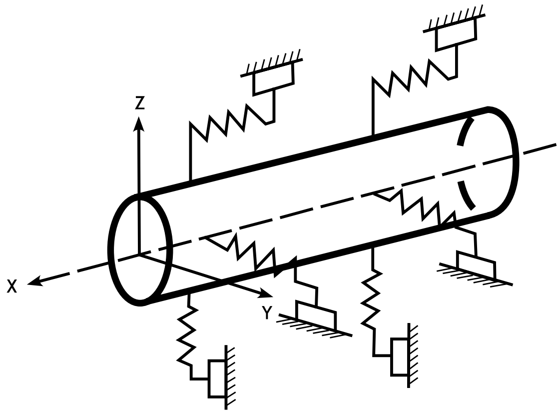
For the pipeline, the key areas of uncertainty relate to predictions of load effects that include pipeline displacement trajectory or path, and local mechanical behaviour such as ovalization, wrinkling, rupture and plastic collapse. The structural beam model does not account for section warping, ovalization or initial load effects (e.g. residual stress or strain). In design, the pipeline load effects are generally accounted for through strength reduction, stress concentration and strain intensity factors (e.g. DNV RP F110, 2007; DNV OS-F101, 2012). The use of an integrated framework through laboratory testing, physical modelling and continuum finite element simulation can establish both serviceability (i.e. normal operations) and ultimate (i.e. strength) limit states (e.g. Fatemi et al., 2012; Kibey et al., 2010).

In frontier regions, such as deepwater and arctic environments, the technical challenges primarily associated with large deformation geohazards, and economic constraints due to limitations of current engineering technologies, provide the needed motivation to advance the state-of-art (e.g. continuum pipe-soil interaction models) to become an integral component within an improved engineering state-of-practice. Successful outcomes can be achieved through an integrated technology framework that includes elements of laboratory testing, physical modelling and numerical simulation (Kenny et al., 2007). This will support the development of practical, safe, reliable and cost-effective design solutions for challenging environments with large deformation geohazards.

This paper discusses elements of this technology framework by examining results from a research program to develop advanced computational simulations tools for assessment of the effects of large deformation ground movement events on buried pipelines. The importance of mesh topology, large deformations and strains, nonlinear material behaviour, and bifurcations on the strategies employed to develop the computational tools are explored.



(a)



(b)

Figure 1. Schematic illustration of the (a) continuum pipe-soil interaction problem and (b) mechanical idealization using beam and spring structural elements

2 ROLE AND VALUE OF PHYSICAL MODELING

As discussed by Randolph and House (2001), there is a pervasive skepticism in geotechnical engineering against relying entirely on results from numerical analysis without the support of experimental data. Accordingly, it is prudent to follow a systematic approach to develop numerical tools for geotechnical problems that have a basis in laboratory testing to define the necessary constitutive parameters and physical modeling to provide a reference dataset for calibration and validation. The laboratory test program will be governed by the chosen constitutive model; however, tests should be conducted over an appropriate stress range as per the expected prototype conditions. As the number of tests are typically limited and may involve complex logistics, the physical test program should focus on addressing the main technical issue with consideration of how best to verify and establish confidence in the numerical simulation procedures. These considerations include thoughtful assessment of soil parameters (e.g. soil type, water content, test bed preparation), test configurations (e.g. geometry, boundary conditions, scale effects), and mechanics (e.g. strain localization, interface behaviour and contact mechanics at the pipe-soil interface). There are also similitude requirements with respect to the pipe and soil mechanical behaviour to ensure the physical model is representative of the prototype (Palmer, 2008; Wang et al., 2012; Wood, 2004).

There are several landmark physical modelling studies (e.g. Audibert and Nyman, 1975; Trautmann, 1983) that have provided significant knowledge on soil behaviour and failure mechanisms during pipe-soil interaction events. These seminal datasets followed a systematic framework, as discussed in the previous paragraph, where they have been successfully used to calibrate and verify numerical tools (e.g. Yimsiri et al., 2004; Badv and Daryani, 2010; Cheong, 2011; Jung, 2011).

Confidence in the verification of the numerical simulation tool is related to the care taken during the physical investigations and quality of the dataset as reported. When considering appropriate datasets to use as a benchmark to help validate numerical tools, it is important to consider potential scale effects, especially when mechanisms such as strain localization are expected. Wood (2004) outlined how potential scale effects result from a) the thickness of ruptures or dilation bands, which is a function of particle size, and b) the mobilization length associated with changing rates of dilation (i.e. the reduction from peak to critical state conditions), which is a function of the relative displacement across the shear band. Where discrete rupture surfaces are formed, with dilation followed by strain-softening, the required ratio of minimum structural dimension to shear band width to achieve an asymptotic response, for full-scale structures such as pipelines or pile foundations, is about 20 (Randolph and House, 2001).

Dense sand exhibits uniform deformation behaviour until a bifurcation point is reached where a shear band develops and the deformation pattern transitions from uniform response to localized strain patterns (Vermeer and de Borst, 1984). Typical dense sand behaviour under direct shear conditions is characterized by four stages: 1) initial quasi-elastic behavior up to a yield point prior to dilation, 2) plastic hardening and increasing dilation to a clearly defined peak, 3) softening behavior associated with shear band formation, and 4) a residual state at which shearing is accumulated along the shear band without any further volume change (Anastasopoulos et al., 2007).

For large scale deformations, the plastic strain magnitude, intensity and gradient can be significant and may exceed the limits of applicability for conventional constitutive models, based on standard laboratory tests and numerical schemes (e.g. performance of the finite element analysis). This issue is also partly addressed by Trautmann (1983). The effect of varying strain is particularly important for dense sands because the peak friction angle may be mobilized at only one point on the failure surface at peak load. At other points on the failure surface, the mobilized shearing resistance is less, and the average mobilized shear resistance, therefore, is smaller than peak at large scales. It has been demonstrated (De Beer, 1963, 1970; Vesic, 1964; Yamaguchi et al., 1976) that this causes large scale prototypes to fail at loads lower than predicted on the basis of small-scale models and dimensional analysis. Physical pipe-soil interaction tests conducted by Hsu (1996, 2001, 2006), Audibert and Nyman (1977) and Trautmann et al. (1985) exhibited a minor effect on soil resistance to lateral pipe movements over a range of practical pipeline diameters, especially in dense sand. Hence, the issue of pipe diameter and model size applies to small-scale models where scale effects cannot be resolved by dimensional analysis.

The influence of pipe burial depth, soil weight and failure mechanisms has been highlighted in recent studies (Phillips et al., 2004b; Daiyan et al., 2010; Rossiter and Kenny, 2012). The contribution of passive soil weight to the lateral resistance for shallow buried pipes was identified, and a modified relationship accounting for this term was proposed (Phillips et al., 2004b) based on the work by Rowe and Davis (1982). The relative contribution was shown to diminish with depth, relating to a transition in the governing failure mechanism. Guo and Stolle (2005) examined the influence of pipe diameter and model size effects, suggesting that small-scale (e.g. < 50 mm pipe diameter) models may be influenced by stress level effects to substantially increase the dimensionless lateral resistance; however, for a practical range of pipeline diameters, there are minor differences in the lateral resistance, consistent with physical observations (Audibert and Nyman, 1975; Trautmann, 1983; Hsu, 2006).

3 NUMERICAL SIMULATION OF PIPE-SOIL INTERACTION

3.1 Finite Element Modelling Procedures

Three-dimensional continuum finite element modelling procedures were developed, using the software package ABAQUS, to simulate the interaction between a rigid pipe and surrounding compliant soil. The pipe and soil were modelled using three-dimensional, 8 noded, reduced integration, C3D8R elements. The Arbitrary Lagrangian Eulerian (ALE) formulation was used in this study. The pipe-soil contact interface was defined by Coulomb interface friction model with penalty method used to account for the effects of relative pipe penetration and over-closure. Boundary conditions were defined to represent two-dimensional plane strain conditions that was consistent with the physical model. Further detailed discussion on the technical requirements, challenges and constraints are presented in several related publications that present the model development and basis (Pike et al., 2012,2013; Pike and Kenny, 2011,2012).

The load conditions were established in two steps, whereby an initial geostatic step established the initial soil stress state, and in the second step, the rigid pipe is displaced laterally. Gravity is applied to the whole model, with the pipe assumed to be in the empty condition (i.e. light), and the pipe does not have any kinematic constraint for movement in the vertical plane.

3.2 Model Calibration and Verification

Trautmann (1983) performed lateral and uplift physical pipe-soil interaction tests at Cornell University (CU). Most of the tests were performed using a 102 mm outer diameter (6.4 mm wall thickness) pipe segment at various burial depths in sand. The sand density was closely controlled to achieve loose, medium and dense conditions, at relative densities of 0, 45 and 80% respectively. The sand, commonly referred to as CU Filter sand, has a specific gravity of about 2.7 (2.74: Trautmann, 1983; 2.69: Robert, 2010) and is classified as poorly graded (Olson, 2009). The mean grain size, d_{50}, is about 0.5 mm on average based on gradation provided by Trautmann (1983) and Olson (2009).

The force-displacement response in very dense sand was characterized by a peak force followed by a softening response; typically, the residual force at large displacement was about 80% of the peak force. Three tests (No. 23, 24 and 25 as referenced by Trautmann, 1983) in dense sand ($\gamma'_d = 17.7\ kN/m^3$) were selected for the calibration study, which evaluated the numerical predictions of pipe-soil load-displacement response, and failure mechanisms with physical test data and observations. The burial depth ratio H/D is defined using the springline burial depth, H, and outer pipe diameter, D, as illustrated in Figure 2.

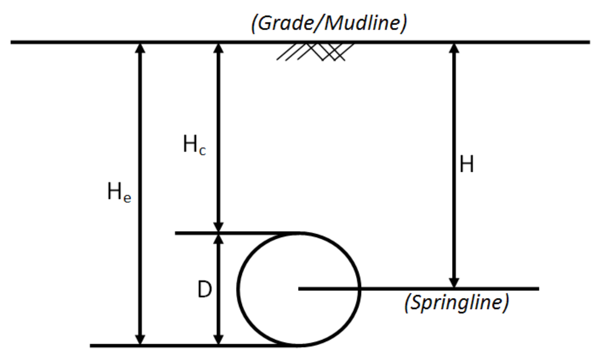


Figure 2. Pipe-soil interaction geometry

3.3 Constitutive Model

The Mohr-Coulomb model available in Abaqus is incapable of reproducing strain softening behavior without modification via user-subroutine. Walters and Thomas (1982) demonstrated that elasto-plastic finite element analyses with strain softening can be used to model shear zone development in sand. However, in order to model initial and subsequent shear zone development, variable non-associated flow rules to control dilatancy were required. Several studies (e.g. Vermeer and de Borst, 1984; Potts and Zdravkovic, 1999; Wood, 2004; Hsu, 2005) have described strain hardening and/or softening Mohr-Coulomb models that improve upon the conventional Mohr-Coulomb model. The variation of strength and volume change parameters with deviatoric strain during hardening and softening can be idealized by linear or exponential relationships that pivot about a peak value.

Mesh or element-size dependency has been encountered when modeling localization in non-associated strain-hardening or strain-softening geomaterials (Needleman, 1988). For a standard continuum, there is a loss of ellipticity of the governing constitutive equations (i.e. bifurcation into the shear band mode coincides with the governing incremental equations having a non-unique solution), leading to element-size dependency, as seen in non-converged shear band widths and load-displacement responses (Yap and Hicks, 2001). Recognizing that strain localization is not a continuum phenomenon but instead is characterized by a length scale, Anastasopoulos et al. (2007) suggested a method of scaling the plastic deviatoric strain corresponding to the start of residual strength and dilation as a function of element size.

Anastasopoulos et al. (2007) proposed a modified Mohr-Coulomb strain-softening model that reduces the mobilized friction and dilation angles (ϕ'_{mob} and ψ_{mob}, respectively) with increase in plastic deviatoric shear strain (γ^p). The mobilized friction and dilation angles are reduced linearly from peak values (ϕ'_p and ψ_p) to residual (critical state) values (ϕ'_{crit} and 0). Hence, the plastic behavior depends on the softening of the yield surface and flow potential based on deviatoric strains.

The plastic shear strain at the end of softening (γ_f^p) is based on direct shear test data. Anastasopoulos (2007) established a relationship to scale γ_f^p as a function of finite element (FE) size and direct shear test data per the following equation:

$$\gamma_f^p = (\delta x_p - \delta x_y)/H_s + (\delta x_f - \delta x_p)/d_{FE} \qquad [1]$$

where H_s is the height of the direct shear test specimen, d_{FE} is the characteristic finite element length, and δx_y, δx_p and δx_f are the horizontal displacements at yield, peak and residual state at which full softening occurs. Anastasopoulos (2007) proposed that the shear strain development during softening can be divided by the ratio between the real shear strain and the FE computed shear strain (γ_{FE}) in order to make γ_{FE} compatible with actual shear strain, thus incorporating length scale effects. The width of the shear band is commonly related to the mean grain size, d_{50}, and is suggested as a suitable characteristic element length. Shear band thickness can range from 8 d_{50} to 20 d_{50} (Jung and Zhang, 2011).

As shown in Figure 3, the present study proposes an enhancement to the above model, to harden the friction and dilation angle from an assumed friction angle, and zero dilation. Without hardening, it is assumed that the response is purely elastic until peak strength is mobilized. Adding the hardening behaviour provides an improvement towards capturing more realistic soil behaviour from the onset of plasticity, as it is generally understood that the dilation angle is approximately zero at first soil yield. The plastic deviatoric strain corresponding to peak mobilized friction and dilation angle, γ_p^p can be determined from the first term of Equation [1].

A user-subroutine (user-defined field) was developed for use with ABAQUS/Explicit that effectively allowed the Mohr-Coulomb parameters to vary with deviatoric strain with incremental loading and deformation. The relationship between the Mohr-Coulomb parameters, as a function of deviatoric strain, is approximated as a piecewise, multi-linear relationship that is defined in tabular format within the numerical procedures.

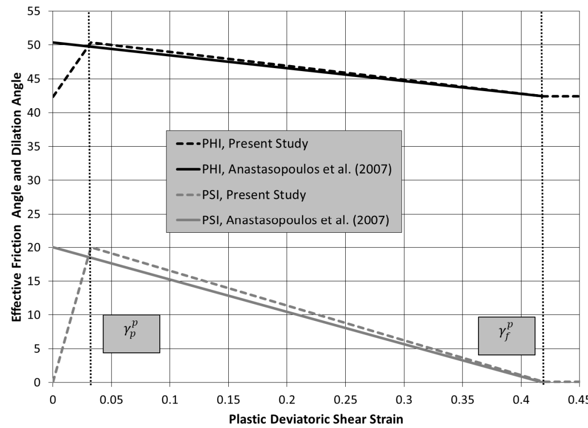


Figure 3. Variation of friction angle and dilation angle

For this study, the constitutive parameters were derived from direct shear tests on CU filter sand that have been presented in the available literature (Trautmann, 1983; Olson, 2009; Robert, 2010). As discussed by O'Rourke (2010), the peak direct shear resistance and applied normal force does not represent a point on the Mohr circle of stress that is tangent to the Mohr-Coulomb failure surface; i.e. does not provide maximum obliquity. The Mohr-Coulomb model requires specification of the maximum obliquity, effective friction angle for plane-strain soil-structure interaction problems. For direct shear box tests, at peak strength the principal axes of stress and incremental strain are coincident; see for example Lings and Dietz (2004) and Bolton (1986). Assuming coaxial response, and considering that the horizontal axis is a direction of zero linear incremental strain (i.e. zero extension), the dilation angle and Mohr's circle of stress can be used to develop a relationship between the direct shear and plane strain values. Davis (1968) first derived an equation linking the parameters as:

$$\sin(\phi'_{ps}) = \tan(\phi'_{ds})/(\cos(\psi)+\sin(\psi)\tan(\phi'_{ds})) \qquad [2]$$

where ϕ'_{ps} is the plane-strain friction angle. At the critical state, when the dilation angle is zero, the equation becomes:

$$\sin(\phi'_{crit}) = \tan(\phi'_{ds\text{-}ld}) \qquad [3]$$

where *ld* denotes large displacement. The critical plane strain friction angle, $\phi'_{ps\text{-}crit}$, is written simply as ϕ'_{crit}.

The internal friction angle determined from direct shear tests was shown to be relatively constant over the stress range of 2.5 kPa to 20 kPa typical of the at-rest soil stresses during the pipe loading tests (Trautmann, 1983). For dense sand, the peak direct shear friction angle ($\phi'_{ds\text{-}p}$) was estimated to be 44°. The ϕ'_{ps} was calculated based on equation [2] using the respective dilation angle, which is discussed in the following paragraphs.

Olson (2009) obtained an estimated average estimated peak plane strain critical friction angle (ϕ'_{crit}) of *38.6°* and demonstrated, for the problems investigated, that the non-associated flow rules linking shear stress and volume change (Taylor, 1948; Rowe, 1969; Bolton, 1986) provided an effective relationship between the direct shear peak friction angle and peak dilation angle when using a constant mean value for the peak critical friction angle. Robert (2010) found that a peak, plane strain critical friction angle (ϕ'_{crit}) of 39°, using Bolton's (1986) equation, provided effective numerical simulations of the physical data for dry sand. This friction angle would equate to a direct shear friction angle at large displacements ($\phi'_{ds\text{-}ld}$) of 32°, which is consistent with the dataset of Trautmann (1983). In this study, a value of 34° was used to define $\phi'_{ds\text{-}ld}$ based on the excellent correspondence between physical model data and numerical simulations, as discussed in Section 3.4.

Robert (2010) presented a significant dataset characterizing peak friction and dilation angles from direct shear tests performed on CU filter sand with varying unit weight and normal pressure. Using this dataset, the present study developed a regression equation to express dilation angle as a function of unit weight and normal pressure (using 10 to 40 kPa values only). Based on a study by Olson (2009), the direct shear dilation measurements at a normal pressure of 22.3 kPa were compared with the regression equation. The direct shear test data for 5 kPa, as reported by Trautmann (1983), were also analyzed to establish peak and residual friction angles. Subsequently, Bolton's (1986) equation, adjusted for direct shear parameters (Lings and Dietz, 2004), was used to represent the dilation angle. Based on this assessment, the regression equation provides a basis to estimate the dilation angle and exhibits high statistical significance with the source data, as shown in Figure 4. Hence, this relationship was used to characterise the peak dilation in the present study for numerical simulation of the Trautmann (1983) tests.

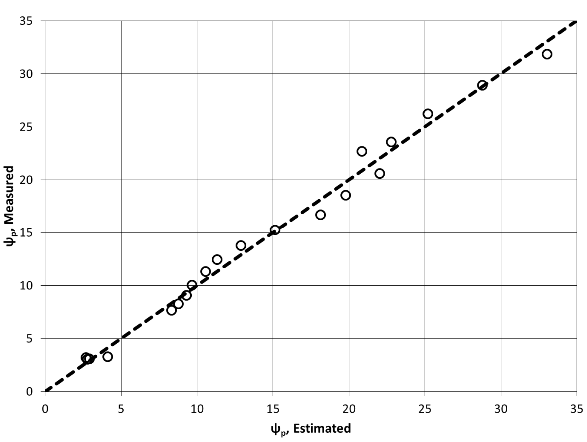


Figure 4. Peak dilation as a function of unit weight and normal pressure, measured vs. estimated values

The elastic deformation properties were varied with depth, as a function of the confining pressure, based on Janbu's (1963) approach. Equation 1 relates the secant Young's modulus, E_s, to the effective confining stress via the power law relationship:

$$E_s/P_a = K((K_o*\gamma'*H)/P_a)^n \quad [2]$$

where P_a is a the reference atmospheric pressure, K_o is the at-rest lateral earth pressure coefficient, γ' is the effective unit weight, H is the soil depth, and K and n are coefficients of the power series fit. Jung and Zhang (2011) determined calibrated coefficients against the Trautmann (1983) tests for dense CU filter sand with $E_s = E_{70}$, $K = 181.25$ and $n = 0.585$.

A similar approach was taken by Cheong et al. (2006) and Robert (2010), except the elastic parameters were defined using the shear modulus, $G = A\ p^n$. Cheong et al. (2006) determined values A = 300 and n = 0.5 from unloading/loading data of drained triaxial tests. Robert's (2010) estimates were higher by a factor of 4.7 with A = 1400 and n = 0.5. Jung and Zhang's (2011) estimates are about 2.5 to 3 times higher than those reported by Cheong et al. (2006). For the present study, the mid-range estimates (Jung and Zhang, 2011) were chosen and a Poisson's Ratio (ν) of 0.3 was used. A small amount of cohesion was also applied for numerical stability, $c' = 0.1\ kPa$.

The specimen height and horizontal displacements corresponding to yield, peak and residual points, per equation [1] were estimated as $H_s = 39mm$, $\delta x_y = 0.53mm$, $\delta x_p = 1.8mm$ and $\delta x_f = 5.3mm$; respectively. A characteristic element length $d_{FE} = 9mm$ gives a ratio of approximately $18\ d_{50}$. Hence, the peak and residual shear strains are estimated as $\gamma_p^p = 0.033$ and $\gamma_f^p = 0.42$. The remaining soil parameter values are summarized in Table 1.

Table 1 Summary of soil parameters

Parameter	Test 23	Test 24	Test 25
H/D	3.5	5.5	8.0
$\phi'_{ps\text{-}y}$ (°)	42.4	42.4	42.4
$\phi'_{ps\text{-}p}$ (°)	49.7	50.4	50.9
ϕ'_{crit} (°)	42.4	42.4	42.4
ψ_p (°)	20.3	19.6	19.0

The geostatic stress state in the soil using ABAQUS/Explicit is limited to initial horizontal to vertical effective stress ratio, $K_o = 1.0$ conditions. However, according to implicit FEA by Rowe and Davis (1982), for vertical plate anchors, and Jung (2011), for buried pipes, K_o has a small effect on lateral soil resistance, for an equivalent range of burial depths. Rowe and Davis (1982) varied K_o from 0.5 to 2.0 and noted less than 10% difference in horizontal resistance, and Jung (2011) noted about 1% difference in peak loads for the same K_o range. The lack of sensitivity to this parameter may be attributed to the large increase in lateral soil stresses due to pipe loading pressures relative to the initial horizontal soil stresses and the more important role of vertical stresses on control of passive and active earth pressures.

The condition of the pipe surface in the Trautmann (1983) pipe-soil interaction tests was described as rough and scaly with minor rust patches. In several studies, the interface friction angle, ϕ_μ, has been approximated as $0.5\ \phi'_{ds\text{-}p}$ (Yimsiri et al., 2004; Badv and Daryani, 2010; Cheong et al., 2011). At H/D of 2, a range of interface friction angles from 0.5 $\phi'_{ds\text{-}p}$ to 1.0 $\phi'_{ds\text{-}p}$ showed a modest increase of 8% in the peak forces (Yimsiri et al., 2004). Olson (2009) estimated $\phi_\mu = 0.6\ \phi'_{ds\text{-}p}$ for smooth steel-sand interaction based on laboratory tests. Hence, for relatively rough steel, it is logical that ϕ_μ should be greater than $0.6\ \phi'_{ds\text{-}p}$; $\phi_\mu = 0.8\ \phi'_{ds\text{-}p}$ was considered appropriate, in this study, for simulating the Trautmann (1983) test conditions.

3.4 Results

For buried pipelines, there are two distinct failure mechanisms associated with lateral pipe-soil interaction that are related to the pipe burial depth (i.e. shallow and deep burial conditions). Depending on soil density, Audibert and Nyman (1975) and Trautmann (1983) observed a distinct soil wedge for the shallow burial depth condition for H/D < 11 and H/D < 8, respectively. Based on numerical simulations, Yimsiri et al. (2004) suggested that shallow burial failure mechanisms existed for H/D < 12 to 16 for loose to dense conditions. The overall soil wedge was comprised of three distinct zones: (1) an almost vertical active zone towards the back of the model pipe, (2) a soil wall above the pipe that extended to the soil surface and (3) a passive wedge bound by a logarithmic spiral in front of the pipe. For extreme cover depths, a punching mechanism extending two to three diameters in loose sand, and approximately one diameter in dense sand, was observed. At shallow embedment conditions, noticeable surface heave was present; however, there were no visible signs of disturbance at the surface for deep conditions (Audibert and Nyman, 1975).

Analysis of the displacement fields in Trautmann (1983) indicates that the extent of the passive wedge ahead of the pipe depends on the soil density and the pipe burial depth. In dense conditions for example, the extent at burial depth ratios of 1.5, 3.5 and 5.5 is approximately 3, 6 and 7 pipe diameters from the final pipe centreline position. This suggests that the ratio of passive wedge extent to burial depth diminishes nonlinearly, suggesting a continual transition to the deep failure mechanism. The simulation of Trautmann's (1983) Test 23 in dense sand at a burial depth ratio of 3.5 accurately produces the extent of the passive wedge (about 6D), as shown in Figure 5.

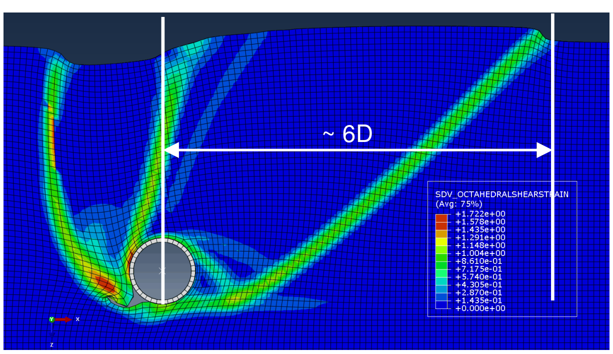


Figure 5. Failure mechanism at H/D = 3.5 in dense sand

As shown in Figure 7, the numerical calculations agree well with the physical dataset at H/D = {3.5, 5.5, 8.0}. The peak dimensionless forces are within +/- 5%, and the softening branch is also captured reasonably well. The ratio of calculated residual to peak forces is from 0.8 to 0.85, consistent with the experimental data. The hardening response is somewhat stiffer in the numerical simulations in comparison to the physical data; a sensitivity analysis on the range of initial elastic modulus estimates has not been conducted, and, as stated by Cheong et al. (2006), lower values may yield a softer response.

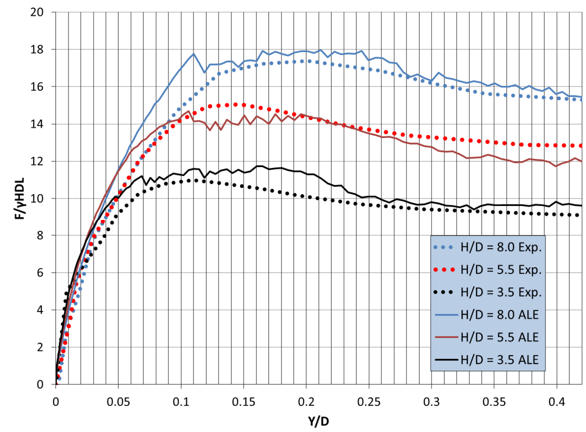


Figure 6. Numerical results vs. Trautmann (1983) experimental data

4 CONCLUSIONS

The technology framework that has a foundation in laboratory testing and physical modeling to provide a basis for calibration and verification of advanced numerical tools for pipe-soil interaction has been outlined and demonstrated. The derivation of constitutive parameters for a modified Mohr-Coulomb model with linear hardening and softening relationships has been described. Using the proposed constitutive model, the numerical finite element simulation tool was shown to provide the physical load-displacement response and expected failure mechanisms very well. The validation of numerical tools that simulate orthogonal pipe-soil interaction builds confidence in modeling procedures for extension to more complex pipe-geohazard interactions such as ground fault rupture (e.g. Xie, 2008; Robert, 2010) or ice keel-pipe-soil interaction scenarios (e.g. Peek and Nobahar, 2012).

An ongoing physical testing program is being carried out by Memorial University, Queen's University and Wood Group to examine stress-strain behaviour, strain localization, load-displacement response and contact mechanics for pipe-soil interaction using laboratory testing and large scale pipe-soil interaction employing particle image velocimetry techniques. The new dataset, developed using the systematic approach described herein will be very valuable towards further validation of the numerical tools developed in parallel with physical testing.

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