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An Effective Stress Analysis of Partially Embedded Offshore Pipelines: Vertical Penetration and Axial Walking

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ABSTRACT

A study on quantifying offshore pipeline resistance during vertical penetration and axial walking is presented, based on coupled pore fluid flow and stress finite element analysis with the Modified Cam Clay model. Following the validation of the numerical method against published centrifuge test results and limit analysis solutions, we present the findings of a detailed parametric study on the response of partially-embedded pipelines under vertical and axial movements, employing 2-D plain strain and full 3-D soil-pipeline models. Emphasis is put on practical findings, and on proposing simplified expressions for the estimation of the contact enhancement factor and of the equivalent friction factor, that can be used at least for preliminary design purposes.

Keywords: offshore pipelines; axial walking; vertical penetration; contact enhancement factor; equivalent friction factor; design

1. INTRODUCTION

Deep-water pipelines are either laid directly on the seabed or are further stabilised through constraining techniques, e.g. trenching, anchoring, or burial. The main objective of such costly constraining measures is to restrict pipeline axial and lateral displacements due to internal and external loads, i.e. temperature fluctuations and high-pressure variations. When the pipeline axial compressive load reaches some critical value, the pipeline becomes unstable and displaces in various directions - vertical (upheaval buckling), lateral (lateral buckling) or axial (pipeline walking) which can all jeopardise its structural integrity. The resisting force developing at the pipe-soil interface directly affects both lateral buckles as well as end expansions. Quantification of this force, both of its initial and steady-state value, is critical for the safe and economical design of pipelines. These aspects of pipeline axial movement have not been investigated in detail, though some remarkable studies have been published recently [1, 2], providing new insight on this multifaceted problem.

A determining factor of pipeline resistance against both lateral buckling and longitudinal expansion is the pipeline embedment, resulting from the dynamic forces during touch-down

and the static submerged weight of the pipeline. Studies of pipeline vertical penetration and embedment in soft clays involve classical plasticity analyses under vertical loading [3, 4] and numerical analyses under combined vertical and horizontal loads with the finite element method [5, 6], accounting for the effect of soil heave [7]. More recently, Martin and White [8] presented exact horizontal-vertical failure envelopes for "wished-in-place" (WIP) pipelines under undrained loading conditions via finite element limit analysis. Furthermore, Krost *et al.* [9] and Chatterjee *et al.* [10] published results of transient finite element analyses using elastic and elasto-plastic soil models, respectively, which focus on contact stress development along the soil-pipeline interface. All the abovementioned studies contribute greatly towards our understanding of the mechanisms of axial resistance development on partially-embedded pipelines (PEPs), which is influenced by the immediate and long-term distribution of contact stress components, an issue that is further investigated in this study.

As mentioned above, Randolph *et al.* [1] proposed a theoretical framework to predict the time-varying axial resistance of pipelines, based on a semi-analytical planar sliding model, which appears to be sensitive to the rate of shear strain, and the elapsed time after initiation of axial movement. The conceptual model proposed by Randolph *et al.* [1], substantiated by experimental results presented by White *et al.* [2], provides a solid basis for describing the mechanisms that govern the frictional behaviour at the pipeline-soil interface of partially-embedded pipelines (PEPs). Yet, as also mentioned by Randolph *et al.* [1], more work is required to derive a universal model for analysing PEPs subjected to vertical and axial displacements, as far as the variation of its input parameters is concerned.

This study aims to shed some light on the mechanisms governing PEPs response during penetration and axial displacement, using a coupled pore fluid flow and stress finite element model (named CFSFE hereinafter for brevity's sake) that takes advantage of the simulation capabilities of ABAQUS/Standard. This model can adequately describe the (slow) axial and vertical movement of pipelines, ignoring inertia effects but accounting for time dependent soil-pipeline interface behaviour, but allowing for the variation of excess pore pressure distribution at the interface and below to be directly taken into account in the estimation of the evolution of the contact normal force with time, which is critical for resistance force predictions.

Accordingly, the presentation can be divided into two parts: The first part focuses on the numerical simulation of vertical pipeline penetration and surface heave generation process with respect to the rate of penetration. Following the validation of the model, we investigate the effects of embedment and frictional contact on the heave development and contact enhancement, and discuss design-related implications of the results. We emphasize particularly on examining the distribution of contact stresses and excess pore pressure development along the soil-pipeline interface, when pipelines are subjected to vertical loads. In the second part, we interpret results of a parametric study for determining the axial resistance of PEPs via a simplified expression, while outlining the factors that govern both the evolution with time and the ultimate value of axial resistance, and emphasising on the limitations that should be considered in pipeline design. In this context, the second part of the study could be considered as an extension of the Randolph *et al.* [1] model, discussing their assumptions in the light of numerical results, and attempting to extend the practical applicability of their conceptual model by presenting the variation of its parameters with the soil properties and the embedment ratio.

2. COUPLED PORE FLUID FLOW-STRESS FINITE ELEMENT MODEL

We consider two different cases in this study, with respect to the pipeline construction procedure: "Wished-in-Place" (WIP) and "Pushed-in-Place" (PIPs) pipelines, referring to pipelines laid on a pre-trenched foundation, or penetrating into the seabed, respectively. To model PIP pipelines, a rigid cylindrical body is pushed into the soil to a target embedment depth, which results in generation of surface heave around the pipeline. For WIP pipelines, the pipe is positioned into its pre-embedment depth, and subsequently a vertical load equal to the submerged weight of the pipeline is applied as an external force. Both a two-dimensional plane strain model and a three-dimensional model are employed to simulate response to vertical and axial movement, respectively. Details on the numerical methodology are presented hereinafter.

2.1. Two-dimensional plane strain model for the simulation of vertical penetration

The geometry and finite element mesh of the 2-D model (Figure 1a) corresponds to the plane strain problem of a pipeline resting on horizontal surface, assuming uniform soil conditions along the length of the pipeline. By taking advantage of symmetry, only half of the problem domain is simulated. We use 8-node reduced-integration pore pressure plane strain elements (CPE8RP) to simulate the foundation soil, whereas the pipeline is modelled as an impermeable rigid body (RB2D2 elements). Parametric runs using full-integration elements (CPE8P) to simulate soil response resulted in trivial discrepancies (less than 3%) in terms of bearing capacity prediction, compared to reduced-integration elements. By using full-integration elements, however, the predicted mode of failure in some cases is influenced by element locking. The reduced-integration formulation of elements with hourglassing control was thus employed in consequent stages of this study, as it exhibits superior performance in similar cases. Simulating the pipeline by rigid elements also does not affect the accuracy of the analyses, since pipeline experiences negligible deformations in comparison to the soft clay.

The surface-to-surface contact formulation is used, with master and slave surfaces being the pipeline and the soil contact areas, respectively. The soil mesh in the contact area is more refined, to avoid significant penetration of the slave surface into the rigid body (master surface) and also to provide more accurate results in the area of interest. Two scenarios were considered in terms of the friction coefficient at the soil-pipeline interface: smooth (frictionless) contact ($\mu=0$) and rough contact ($\mu=\infty$), where no slippage at the soil-pipeline interface is permitted. The frictional response of the contact interface is defined via the classical Coulomb friction model. Furthermore, the common "penalty" method of contact enforcement is employed to evaluate normal pressure at the contacting surfaces.

Effect of mesh size on the finite element predictions was addressed via a mesh sensitivity analysis, which was carried out to identify the optimum mesh density for the soil element domain. The results of this analysis are not presented here, to maintain the limits of the presentation.

Figure 1. Finite element mesh and geometry of the (a) two-dimensional plane-strain model, (b) three-dimensional model.

The embedment ratio $E_R = w/D$ defines the initial embedment of the pipeline into the seabed (Figure 2), where w is the initial depth of embedment and D is the diameter of the pipe, here $D=0.8\text{m}$. The pipeline is placed sufficiently far from the lateral and bottom boundaries of the model. Initial pilot analyses have shown that placing the lateral boundaries at distance equal to $4D$ from both sides of the pipeline, and the bottom boundary at a distance equal to $5D$ from the bottom of the pipeline (Figure 1a) is sufficient to eliminate boundary interaction effects. Drainage is allowed only through the top boundary of the mesh during all analysis steps, by setting the total pore pressure equal to zero, while all other boundaries are modelled as impermeable.

Figure 2. Schematic diagram of PIP pipeline installation resulting in surface heave.

Each analysis is run in three distinct steps. An initial geostatic step ensures equilibrium conditions. During this initial step, the hydrostatic pore pressure at the top boundary of the finite element mesh is assumed to be zero. Once the equilibrium is achieved, a vertical load equal to the pipeline submerged weight (per unit length) is applied on the WIP pipeline, with the duration of this step being 1 sec. Alternatively, for the PIP pipeline case, the rigid pipe is pushed downward by applying a particular displacement, w , increasing from zero to its

ultimate value within 86400 sec (1day). This duration was selected so as to effectively cover both undrained and fully-drained response, in conjunction with the varying permeability of the seabed soil, as discussed in the following sections. This type of displacement-controlled analysis allows for the effect of soil heave to develop, and here $E_R=w/D$ is the penetration depth.

Once the pipeline self-weight is applied, during the second step of the analysis, the soil surrounding the pipeline is deformed, and excess pore pressure is generated. Time duration for the third, consolidation step is long enough so that the full consolidation settlement develops, and excess pore pressures dissipate. The initial time increment of the consolidation step is critical for the convergence of the solution, and is determined by the Vermeer and Verruijt [11] criterion, as:

$$\Delta t_{initial} = \frac{h^2 \gamma_w}{6E'k} \quad (1)$$

where h represents the average element dimension, k is the soil permeability, and E' is the effective Young's modulus of the soil. The NLGEOM parameter is set to on during all analyses, to account for geometric nonlinearity effects in the solution.

2.2. Three-dimensional model for the simulation of axial walking

The three-dimensional geometry and mesh used to simulate the axial resistance of PEPs are shown in Figure 1b. Apart from the geometry, the general analysis concepts described above are integrated in the three-dimensional model as well. We use 20-node reduced-integration pore pressure brick elements (C3D20RP element type) to discretise the soil mass, and rigid shell elements (R3D4 type) for the pipeline. Pilot analyses proved that using reduced-integration elements with hourglassing control results in significantly lower computational cost at no expense of accuracy, as discussed before. The soil body has a unit width (1m), which is one-fifth of the simulated pipeline length, to ensure its contact with the soil all through the axial walking step. The axial walking is simulated by imposing an axial displacement to all pipeline nodes, increasing from zero to $1.4D$ over a duration of 864000 sec (10 days). During this step, the vertical displacement of the pipeline nodes is not constrained.

2.3. Material model and properties

The popular Modified Cam Clay (MCC) model is used in this study, with the parameters listed in Table 1. Two distinct sets of parameters with varying permeability are employed. The first set (MCC1) resulted from the back-analyses of centrifuge experiments of pipeline vertical penetration presented by Dingle *et al.* [12], and is used in the relevant part of the study. The second set of parameters is used during the second part of the study, on the axial friction estimation. The merit of using different sets is to investigate their effects on the axial resistance of the pipeline; as Randolph *et al.* [1] report that the calibration of the input parameters of their semi-analytical model was based on particular soil properties, we will try here to investigate the effects of the constitutive model parameters on axial resistance prediction.

Table 1. MCC parameters used throughout this study.

Parameter	MCC1	MCC2	Unit
Slope of virgin consolidation line, λ	0.4	0.3	-
Slope of recompression/swelling line, κ	0.115	0.05	-
Slope of critical state line, M	1.8	1.2	-
Initial void ratio, e_0	1.5	1.8	-
Poisson's ratio, ν	0.333	0.25	-
Coefficient of lateral earth pressure, K_0	0.8	0.8	-
Dry density, γ_{dry}	11.0	10.0	kN/m ³
Permeability, $k_v = k_h$	varying	varying	m/s

Although the MCC model is described in detail in many textbooks, we are discussing hereinafter certain aspects of the model used later in this study for the presentation of the results. The undrained shear strength for anisotropically consolidated MCC soil under plane strain conditions is estimated as [13]:

$$s_u = \frac{\sin \varphi'}{2a} \left(\frac{a^2 + 1}{2} \right)^A \sigma'_{v0} \quad (2)$$

where $a = \frac{3 - \sin \varphi'}{2(3 - 2 \sin \varphi')}$, $A = 1 - \kappa/\lambda$ and φ' is the critical-state friction angle of the soil.

For the general three-dimensional model, undrained shear strength can be expressed as:

$$s_u = \frac{1}{2} M \left(\frac{e_{cs} - e_0}{\lambda} \right) \quad (3)$$

where:

$$e_{cs} = e_1 - (\lambda - \kappa) \ln 2 \quad (4)$$

$$e_1 = e_0 + \kappa \ln p'_0 + (\lambda - \kappa) \ln p'_c \quad (5)$$

and p'_c is the mean normal pre-consolidation stress; p'_0 is the initial mean effective stress. The isotropic overconsolidation ratio R is defined as the ratio of the pre-consolidation stress p'_c to current mean normal effective stress p' :

$$R = p'_c / p' \quad (6)$$

For a K_0 -anisotropically consolidated MCC soil, the isotropic overconsolidation ratio R can be linked to the conventional overconsolidation ratio $OCR = \sigma'_{v0} / \sigma'_c$ through the following equation [14]:

$$R = \alpha_R \cdot OCR \quad (7)$$

with α_R being a function of φ' and of the soil stress history, σ'_{v0} being the initial and σ'_c the current vertical effective stress. Furthermore, based on Terzaghi's one-dimensional consolidation theory, hydraulic conductivity of a soil is associated to the coefficient of consolidation, c_v as:

$$c_v = \frac{k_v}{m_v \cdot \gamma_w} \quad (8)$$

where m_v is the coefficient of volume compressibility, k_v is the vertical permeability; and γ_w is the density of the pore fluid. In the MCC model, changes in volumetric strain are expressed as:

$$d\varepsilon_v = \frac{\lambda}{1+e_0} \left(\frac{dp'}{p'} + \left(1 - \frac{\kappa}{\lambda}\right) \frac{2\eta d\eta}{M^2 + \eta^2} \right) \quad (9)$$

where $\eta = q/p'$, q being the deviatoric stress and the coefficient of soil compressibility being:

$$m_v = \frac{1}{1+e_0} \frac{\Delta e}{\Delta \sigma'_v} \quad (10)$$

By assuming that the in-situ coefficient of earth pressure K_0 remains constant with changes in the stress state of an anisotropically consolidated clay:

$$\frac{\Delta p'}{\Delta \sigma'_v} = \frac{(1+2K_0)}{3} \quad (11)$$

By combining (9), (10), and (11), and replacing $\Delta e/(1+e_0)$ with $d\varepsilon_v$, the coefficient of volume compressibility can be expressed as:

$$m_v = \frac{\omega}{3(1+e_0)} \frac{(1+2K_0)}{p'} \quad (12)$$

where ω is a consolidation-state parameter and depends on the stress state of the soil during consolidation; in other words, whether the soil element is normally consolidated or over-consolidated, when excess pore pressure dissipates. Parameter ω attains values between the compression index and the swelling index $\kappa < \omega < \lambda$. For a normally consolidated to lightly-overconsolidated soil experiencing isotropic consolidation, we can replace ω with λ . As a result, the coefficient of consolidation of a MCC soil can be related to its vertical permeability through the following equation:

$$(c_v)_{NC} = \left[\frac{3(1+e_0)}{(1+2K_0)} \right] p' \left(\frac{k_v}{\lambda \gamma_w} \right) \quad \text{if } R=1.0 \quad (13)$$

Eq. (13) is derived on the basis of a few simplifying assumptions, such as constant lateral earth pressure coefficient; constant permeability with changes in stress; soil volume change being a result of changes in the volume of voids only; Terzaghi's uncoupled consolidation theory is valid. It can be induced from eq. (13) that the coefficient of consolidation of a MCC soil is not constant, and varies with mean effective stress. For simplicity, in this study we will refer to the initial value of consolidation coefficient when normalizing results i.e. the value corresponding to the initial mean effective stress.

2.4. Numerical model validation against previous studies

In the following we present the validation of the numerical methodology employed in this study (CFSFE model) against results from the literature for the vertical penetration problem, both for PIP and WIP pipelines. Comparison is provided in terms of resistance-displacement curves, and heave profile prediction.

Focusing first on resistance development with increasing penetration, we compare in Figure 3a normalized load-displacement curves derived with the CFSFE model against the centrifuge test results presented by Dingle *et al.* [12]. A similar comparison is attempted in Figure 3b with several analytical and numerical studies: theoretical plasticity solutions [4, 8];

curve fits of numerical results proposed by Merifield *et al.* [7] for smooth and rough pipes; and numerical results obtained by Zhou *et al.* [15] while using an extended Tresca model. Resistance curves are plotted up to pipeline penetration depth $w=0.45D$, while two "extreme" cases are considered with respect to the pipeline-soil interface properties: that of smooth pipeline ($\mu=0$) and that of rough pipeline ($\mu=\infty$), as discussed earlier.

Figure 3. Comparison of resistance-displacement curves for (a, b) PIP and (c) WIP pipelines.

The undrained shear strength profile of the centrifuge tests, used for the calibration of the model, is expressed as [13]:

$$\left(\frac{s_u}{\sigma'_{v0}}\right) = \left(\frac{s_u}{\sigma'_{v0}}\right)_{NC} (OCR)^\Lambda \quad (14)$$

where the back-calculated values of $s_u/\sigma'_{v0}=0.19$ and $\Lambda=0.71$ are reported by Dingle *et al.* [12]. The model parameters (set MCC1, Table 1) were selected on the basis of the undrained shear strength profile and physical characteristics reported by Dingle *et al.* [12]. Initial void ratio and in-situ earth pressure coefficient K_0 were subsequently determined via back-analysis of centrifuge test results, so as to achieve the best possible match in terms of soil resistance. A surcharge pressure of 7 KPa is applied on the top boundary of the mesh including the segment underneath the pipeline, to establish consistency with the reported shear strength profile. Penetration resistance values are normalized against the undrained shear strength s_u at the pipe invert.

Focusing first on PIP pipelines, notice in Figure 3a that the experimental penetration resistance is somewhat higher, compared to the CFSFE numerical predictions for smooth pipeline, for shallow penetration depths ($E_R=w/D<0.18$). On the other hand, numerical predictions for a smooth pipe demonstrate a closer match to the centrifuge data for penetration depths ranging from $0.18<E_R<0.35$. The CFSFE prediction is fairly comparable with the results of Zhou *et al.* [15] using an extended Tresca model. Plasticity solutions noticeably underestimate the penetration resistance; these upper bound solutions refer to a WIP pipeline resting on weightless soil. Finite element predictions by Merifield *et al.* [7] consider both the effect of soil heave and self-weight in the penetration resistance profile, but simulate soil response via a simple elastic-rigid plastic model.

The second part of the verification of the CFSFE model refers to WIP pipelines. For that, we will use the results by Martin and White [8] which present finite element limit analyses for smooth and rough WIP pipelines, for linearly increasing undrained shear strength with depth. For comparison purposes, five (5) analyses were performed for a WIP pipeline embedded at characteristic depths ranging from $E_R=0.1$ to $E_R=0.5$. The undrained shear strength gradient k_s (kPa/m) corresponding to MCC1 model parameters is $k_s=3.6$ kPa/m, whereas the submerged weight-to-strength ratio is equal to $\gamma'/k_s=1.4$. Martin and White [8] presented results both for unlimited and zero interface tensile capacity; herein we compare results with the latter case. Note that the study at hand focuses on partially-embedded pipelines, thus embedment ratios $E_R \leq 0.5$; Martin and White's study extends to embedment ratios up to $E_R=5$. Notice from Figure 3c that the CFSFE model results are compatible with the results of the limit analyses, both for the smooth and rough pipeline case. We have to mention here however that the results by Martin and White [8] correspond to $\gamma'/k_s=1$; Martin and White did not present penetration resistance curves for zero tensile capacity and $\gamma'/k_s > 1$.

Further evidence of the validity of the model is sought via comparing predictions of heave development with experimental observations reported by Dingle *et al.* [12] and numerical results by Merifield *et al.* [7]. Heave is of significant interest to pipeline design, as it introduces additional resistance against both axial and lateral pipeline displacements, mainly due to the increase in the pipeline embedment depth. Further to that, heave also enhances penetration resistance by a fraction, due to the additional surcharge load imposed by the heaving soil. Merifield *et al.* [7] presented a semi-analytical method for determining the surface heave profile, by idealising the generated heave as a rectangular block of width $0.8D'$, with D' being the contact width (Figure 2). By assuming a fully undrained state of pipeline penetration, the height of the hypothetical rectangular soil block forming heave on both sides of the pipeline h_{heave}^* equals $V_{swept}/2(0.8D')$, where V_{swept} is the volume of the soil swept away by the penetrating pipeline. Heave generation increases the embedment depth to $w_{heave}=w+h_{heave}^*$ leading to higher effective embedment width, $D'_{contact}$ and longer effective contact perimeter, $p_{contact}$ (Figure 2). The contact width and contact perimeter for the case of a pipeline surrounded by heaved soil are estimated as:

$$D'_{contact} = 2D \sqrt{\frac{w_{heave}}{D} - \left(\frac{w_{heave}}{D}\right)^2} \quad (15)$$

$$p_{contact} = D \cos^{-1} \left(1 - 2 \left(\frac{w - h_{heave}^*}{D} \right) \right) \quad (16)$$

As depicted in Figures 4a and 4b, where numerical results are compared with the experimental results of Dingle *et al.* [12] and the solution of Merifield *et al.* [7], neither the CFSFE analysis nor the semi-analytical solution are able to accurately predict the effective contact width $D'_{contact}$ and the effective contact perimeter, $p_{contact}$ for embedment ratios smaller than $E_R=0.10$. Disturbance of the slurry at the mudline intercept during initial penetration of the cylinder can generate very loose soil heave, which is captured by the image analysis [12], thus resulting in higher $D'_{contact}$ and $p_{contact}$ values. Such disturbance cannot be addressed via similar numerical analysis or analytical models, and accordingly lower effective contact perimeter and contact width are evaluated for $E_R < 0.10$. Note here that the numerically derived contact perimeter and contact width are adjusted to represent those segments of interface under positive contact pressure; local separations at the contact area (with zero contact pressure) are deduced from the total contact length.

Numerical analyses are able to simulate the experimentally measured increase in contact perimeter with respect to embedment depth (Figure 4b), particularly when considering a

smooth pipe ($\mu=0$) for embedment ratio higher than $E_R=0.1$. Finite element predictions for the smooth pipeline case, both from this study and Merifield *et al.* [7], diverge from those for the rough pipeline, which is an indication that the actual failure mechanism and displacement development at the vicinity of a penetrating pipeline is indeed affected by the frictional behaviour of the interface. It must be noted however that results for the smooth pipeline case present a better match to the experimental results.

Figure 4. Comparison of semi-analytical (Merifield *et al.* [7]) and numerical predictions of the (a) normalized contact width and (b) normalized contact perimeter along the soil-pipeline interface, with the experimental results of Dingle *et al.* [12].

In the light of the above, it is not surprising that the numerically predicted heave profile considering smooth interface matches better the experimental results (Figure 5). Introducing a rough interface restricts the movement of soil elements at the pipeline-soil interface, and accordingly disrupts the development of the failure mechanism during pipeline penetration [16]. This explains why for a particular penetration depth, shorter contact perimeter and smaller heave are expected around a rough interface. The phenomenon is more evident for pipeline penetration deeper than about $E_R=0.2$, when the effect of embedment depth on the magnitude of contact perimeter is more significant (Figure 4b). Note that all the analyses presented in this section (including results in Figure 4 and Figure 5) correspond to a fully undrained mode of penetration, so as to be comparable with existing studies employed in the validation of the methodology.

Figure 5. Comparison of numerical and experimental [12] surface heave profiles.

3. PARAMETRIC INVESTIGATION AND PRACTICAL FINDINGS

In the following part of this study, we investigate rate effects on the partially embedded pipeline response during vertical penetration and axial walking. Two main time-related parameters are introduced: the rate of loading (velocity), and the rate of pore pressure dissipation (permeability). Emphasis is put on practical implications resulting from the interpretation of the parametric analysis.

3.1. Drained and undrained response during vertical penetration

Figure 6 demonstrates how the resistance of a PIP pipeline evolves with the normalised penetration velocity, vD/c_v , where v is the penetration velocity and c_v is the coefficient of consolidation in the vertical direction, defined in eq. (13). The particular results correspond to pipeline pushed up to a depth of $E_R=0.45$. Parametric analyses were performed, with the normalized penetration velocity ranging from 8.33×10^{-4} to 8.33×10^6 . In Figure 6, time t is defined via the time factor T , normalized against the coefficient of consolidation c_v and the diameter of the pipeline, D [1]. Vertical penetration resistance V is normalised using the initial undrained shear strength at the level of pipe invert s_{uo} (eq. 2) and the pipe diameter D [4].

The evolution of the penetration resistance with time is derived for each particular velocity, and a backbone curve can be drawn (dashed line in Figure 6), linking all penetration resistance limits. This backbone curve identifies three areas corresponding to the upper and lower bound of penetration resistance, as well as a transition zone. These areas also correspond to three different modes of pipeline penetration: undrained, partially drained and fully drained penetration. For a particular soft clay, the lower bound of penetration resistance is associated with undrained penetration, while the upper bound with fully-drained conditions. This transition from fully drained to fully undrained axial resistance limits is very similar to the variation of piezocone penetration resistance with the penetration rate and soil consolidation rate [17, 18].

Figure 6. Vertical pipeline resistance as a function of drainage conditions.

The zones identified in Figure 6 are also associated with the potential for heave development around the pipeline. For the undrained zone, the volume occupied by the embedded pipe equals the volume of the heaving soil (Figure 7a). Conversely, for the zone corresponding to fully-drained penetration, no heave is developed whatsoever (Figure 7c). In the transition zone labelled as partially drained, only a fraction of the volume occupied by the embedding pipeline will develop as heave, while the remainder will cause soil settlement (Figure 7b). It is clear from the above that fully undrained pipeline penetration must be ensured, prior to including the full effect of the surface heave into design considerations. Figure 6 can act as a guideline for determining the required penetration velocity to achieve that.

(a)

(b)

Figure 7. Numerical surface heave predictions for (a) undrained, (b) partially-drained, and (c) fully drained penetration (embedment ratio $E_R=0.45$).

3.2. Drained and undrained response during axial movement

3.2.1. Contact stresses and excess pore pressure distribution at the soil-pipeline interface, and derivation of the contact enhancement factor ζ

One of the key parameters that affect the magnitude of the friction force resistance to axial walking is the developing contact stresses at the soil-pipeline interface, which are examined in the following. Figure 8a presents the total contact stresses developing at the soil-WIP pipeline interface immediately after application of the pipeline self-weight load, for six (6) different embedment ratios. Results are obtained using the 2-D model and a smooth ($\mu=0$) soil-pipeline interface. To maintain compatibility with previous studies [9, 10], contact stress values are normalised against a mean normal stress q_b , related to the submerged weight of the pipe $q_b=W/D'$, with D' being the pipeline effective width defined in Figure 2. Contact stresses σ_n at the soil-pipeline interface are not directly proportional to the magnitude of the vertical pipeline load, W' , thus normalising the contact pressure against the mean normal stress does not eliminate the effect of the pipeline load on the contact stress distribution profile.

Figure 8. Distribution of (a) total contact stresses and (b) excess pore pressure along soil-pipeline interface (results for normalized velocity $vD/c_v=8.33$).

In the low embedment ratio $E_R=0.1-0.2$ cases, the contact stress exhibits a maximum at the pipe invert and decreases monotonically to practically zero towards the boundaries (Figure 8a), due to negligible confinement at the edges of the pipe i.e. the distribution is similar to the stress distribution below a rigid strip footing on sand. As the embedment ratio increases, the contact pressure at the pipe invert is reduced, while its maximum shifts towards the exterior boundaries, resembling the contact stress distribution below a rigid strip footing on saturated clay. For example, in the case of embedment ratio $E_R=0.5$, the normalised contact pressure σ_n/q_b becomes maximum at a relative distance $x/D'=0.1$ from the boundaries of the pipe and

reduces towards the invert to a value of $\sigma_n/q_b=0.11$. As expected, the contact stress distribution for a hypothetical case of a rigid strip footing, plotted for comparison purposes, is more or less uniform, with the maximum stress being equal to $\sigma_n/q_b=1.0$ underneath the footing axis. Similar results are observed for the distribution of the normalised excess pore pressure $(u-u_0)/q_b$ at the soil-pipeline interface immediately after application of the load (Figure 8b).

Certain discrepancies exist between the contact stress distribution plotted in Figure 8 and the distribution profiles obtained by Krost *et al.* [9] via elastic numerical analyses, which suggest that the contact stress at the invert of the pipeline increases with increasing embedment ratio. Analyses with the non-linear CFSFE model revealed that such contact distribution profile develops only when the contact stresses due to the “vertical load”, i.e., the submerged weight of the pipeline, reach the soil bearing capacity, for the particular pre-embedment depth.

The normal contact stresses at the pipe-soil interface depicted in Figure 8 correspond to a contact force which resultant magnitude generally exceeds the pipe submerged weight W' , due to the non-vertical components of normal contact stresses acting on the periphery of the embedded segment of pipeline. The ratio at which normal contact force resultant magnitude N exceeds the pipeline submerged weight, W' , is referred to in the literature as "wedging factor" or "contact enhancement factor" [19, 20], and it is denoted by ζ . By considering vertical equilibrium and approximating the normal contact stress with a uniform line load distributed along the pipe periphery as a function of $\cos\theta$, where $\theta=\cos^{-1}(1-2E_R)$, White and Randolph [20] derived an expression to calculate the contact enhancement factor at 100% consolidation:

$$\zeta = \frac{N}{V} = \frac{2 \sin \theta}{\theta + 0.5 \sin 2\theta} \leq 1.27 \quad (17)$$

where the averaged normal contact stress along the contact perimeter length, p is:

$$s_n = \frac{zV}{p} \quad (18)$$

In reality however, the normal contact stresses vary non-uniformly along the soil-pipeline interface (Figure 8). Averaging the highly non-uniform normal contact stresses along the pipe-soil interface will introduce some inaccuracy. Furthermore, the contact effective stress do not vary only spatially along the soil-pipe interface, but also vary temporally: the effective contact stress develops a minimum at the time when the pipeline is laid on the seabed, and increases as excess pore pressures dissipate over time. This increase of the normal contact stress with time has been considered by Chatterjee *et al.* [10] through averaging the excess pore pressure along the pipe periphery. Yet, as the distribution of the generated pore pressure at the soil-pipe periphery is very similar to the non-uniform distribution of the normal contact stress (Figure 8b), averaging excess pore pressure over the contact perimeter to estimate the effective contact force would also introduce some inaccuracy, the importance of which is examined hereinafter.

Figure 9. (a) Contact stress components at the pipeline-soil interface and (b) contact effective stress derivation using the CFSFE model results ($E_R=0.5$).

In this study, an alternative approach is proposed to estimate the contact enhancement factor: Using the CFSFE model, the contact effective stress is computed by deducting the excess pore pressure from the total contact stress along the soil-pipeline interface. If σ'_n is the effective contact stress acting on the soil-pipe interface, the normal contact force N' can be derived by integrating σ'_n over the pipe-soil contact perimeter p (Figure 9a). The effective contact stress acting on the embedded part, at any particular time after pipe laying, is calculated as:

$$\sigma'_n = \sigma_n - \Delta u \quad (19)$$

and the contact enhancement factor ζ is equal to:

$$\zeta(t) = \frac{\int_0^t \int_{\theta_1}^{\theta_2} \sigma'_n(t) \cdot r d\theta dt}{\int_0^t \int_{\theta_1}^{\theta_2} \sigma'_{n,y}(t) \cdot r d\theta dt} \quad (20)$$

where $\sigma'_{n,y}$ and σ'_n are the vertical and normal components of contact stress, respectively (Figure 9a).

Both normal contact stress and excess pore pressures can be derived numerically for each contact point on the soil-pipeline interface, at any given time, and effective stress can be estimated as shown in Figure 9b. Thus, based on the analyses results we calculate the contact enhancement factor from eq. (20), and we present its variation with the embedment ratio E_R in Figure 10a, for consolidation ratio values equal to 50% and 100%. For the WIP pipeline case considered first (Figure 10a), the CFSFE results show compatible trends with previous studies [9, 20], though discrepancies exist: contact enhancement factor values are lower by 10% to 15% in comparison to the simplified analytical solution of White and Randolph [20] and the elastic analysis results of Krost *et al.* [9], respectively.

Smaller contact enhancement ratio predictions can be partially attributed to the possible separation at the pipe-mudline interface, specifically for shallow pre-embedment ratios, captured within the present non-linear model. Averaging contact stresses along the pipeline-soil interface rather has a relatively trivial effect: for the specific case in Figure 10a, averaging the contact normal stress along the pipeline periphery results in a higher contact enhancement factor value of about 2% to 5%, compared to the presented values. As expected, the enhancement factor tends to $\zeta=1.0$ as E_R tends to zero; as it would be the case for a strip footing resting on the soil surface.

Figure 10. Predictions of contact enhancement factor (a) comparison with previous studies for the case of a smooth WIP pipe, and (b) effect of installation method and surface roughness.

Increase in the contact perimeter through development of soil heave around the pipeline is numerically investigated in this study also for the PIP pipeline case, considering again both smooth and rough interface. As depicted in Figure 10b, effect of embedment ratio on the contact enhancement factor for WIP pipelines does not depend on the interface properties, while this is not the case for PIP pipelines. In addition, the contact enhancement factor ζ for PIP pipelines is up to 20% higher compared to WIP pipelines, with the difference being more prominent for the smooth interface case. Design guidelines [21] suggest that friction factor at the soil-pipeline interface generally varies between $\mu=0.2$ to 0.7 with an average of $\mu=0.5$. Thus, for design purposes, we propose to conservatively estimate the contact enhancement factor ζ while considering the data for rough PIP pipelines.

We can further apply a linear fit to provide the variation of the contact enhancement factor with the embedment ratio at the end of the consolidation period (consolidation ratio 100%), as:

$$\begin{aligned} \zeta &= 0.618(E_R) + 0.932 \geq 1.0 && \text{for a WIP pipeline} \\ \zeta &= 1.056(E_R) + 0.894 \geq 1.0 && \text{for a PIP pipeline} \end{aligned} \quad (21)$$

The contact enhancement effect can be influential for the design of pipelines with higher embedment ratios, yet we propose to neglect it for $E_R < 0.1$.

3.2.2. Rate effects on axial resistance development

Randolph *et al.* [1] recently proposed an advanced conceptual model of pipeline-soil axial response. Assuming planar shearing conditions, the generation and subsequent dissipation of shear-induced excess pore pressures within a hypothetical thin shear band is linked to the volumetric strain through the critical state framework. The Randolph *et al.* model, which is a noteworthy contribution towards the description of pipeline response during axial walking, currently applies to a specific pipeline embedment ratio ($E_R=0.4$), and is calibrated via numerical analyses on a particular set of soil parameters; thus is of rather limited practical interest. Here we will try to extent the applicability of the framework via a parametric analysis with the CFSFE model. For that we consider a rigid pipeline, pre-embedded (WIP) in a body of soil of unit length, at a normalized depth ranging from $E_R=0.1$ to 0.5 (Figure 2b). As mentioned earlier, the submerged weight of the pipeline is applied in a separate step, after

the initial geostatic step, and a Biot-consolidation step follows. Once the consolidation step is completed, an axial displacement of $1.4D$ is applied to the rigid pipe. Two different sets of MCC parameters (Table 1) are considered, to quantify their effect on their results.

Lets focus first on a series of analyses performed for embedment ratio $E_R=0.4$ i.e. the same as the one considered by Randolph *et al.* [1], employing the MCC2 set of model parameters, to investigate the effect of soil properties on the results. To quantify the mobilized axial friction, we use the equivalent friction factor, defined as the ratio of the axial resistance F over the pipeline submerged weight, W' [1]. In analogy to Figure 6, Figure 11 provides the variation of the equivalent friction factor with the normalised velocity vD/c_v , and normalised time $T^*=c_v t/D^2$ for soil-pipeline interface friction coefficient $\mu=0.3$. Contrary to the case of vertical penetration, the velocity v here is the axial walking velocity. Notice in Figure 11 that the equivalent friction factor F/W' ranges between the lower bound value of $F/W'=0.12$ and $F/W'=0.378$, which correspond to undrained and fully drained states of shearing, respectively. As in the case of normalized vertical shear resistance (Figure 6), a transitional zone corresponding to the partially drained state of shearing links these bound values, and a backbone curve can be identified. Experimental measurements reported by White *et al.* [2] also resulted in considerable variations in the equivalent friction factor for different axial sweep rates, where the difference between equivalent friction factors for slow and fast sweeps reached a ratio larger than 7.0.

Figure 11. Mobilized friction for various axial pipeline velocities.

Qualitatively similar results are provided by Randolph *et al.* [1], although the soil parameters that they consider are different to the parameters of this set of analyses. Randolph *et al.* proposed a fitting equation to describe the backbone curve providing the equivalent friction factor F/W' versus T^* :

$$\frac{F}{W'} = \left(\frac{F}{W'} \right)_{\text{drained}} - \left[\left(\frac{F}{W'} \right)_{\text{drained}} - \left(\frac{F}{W'} \right)_{\text{undrained}} \right] e^{-\ln(2) \left(T^*/T_{50} \right)^m} \quad (22)$$

Where T_{50} is a fitting parameter corresponding to the time at which the frictional resistance is midway between the undrained and drained limits. For the material properties considered in their study, Randolph *et al.* [1] used $T_{50}=0.05$ and $m=0.5$ as fitting parameters. Results of a parametric study that we performed for two different sets of MCC parameters (Table 1)

suggest that values of T_{50} and m introduced in eq. (22) are insensitive to the embedment ratio E_R , but do however depend on the properties of the soil. For the MCC2 set the best-fit parameters are found to be $T_{50}=0.03$ and $m=0.7$ (Figure 12a) and for the MCC1 parameters the corresponding parameters are $T_{50}=0.008$ and $m=0.8$ (Figure 12b). Fair agreement of eq. (22) with the numerical results is noticed, regardless of the embedment ratio (continuous lines in the Figures 12a and 12b). This implies that the general shape of the backbone curve can be efficiently described by eq. (22), but proper fitting of parameters T_{50} and m is required. Figure 13 presents three backbone curves derived for the different MCC parameter sets, for $E_R=0.4$. It is clear that the extent of the partially-drained area of response depends on the parameters of the model considered, and Figure 13 can be used as a guide to evaluate the expected range of this area.

Figure 12. Equivalent friction factor F/W' versus T^* curves for various embedment ratio E_R values and different MCC parameter sets.

Figure 13. Equivalent friction factor F/W' versus T^* curves for different MCC parameter sets.

An approximate expression (dashed lines in Figure 12c-12d) can be derived for estimating the equivalent friction factor F/W' as a function of only the embedment ratio E_R and normalised time T^* :

$$\frac{F}{W'} = e^{-0.9(E_R/0.4)^{-0.8}} - e^{-\left[1.48(E_R/0.33)^{-0.6} + \ln(2)\left(T^*/T_{50}\right)^m\right]} \quad (23)$$

This approximate expression compares fairly well with the exact numerical results, and when considered together with the fitting parameter sets shown in Figure 13, can provide a first estimate of the expected range of equivalent friction factor. It is underlined that this expression applies to the specific value of the interface friction coefficient used $\mu=0.3$; different interface coefficient factors will result in different drained and undrained equivalent friction factor F/W' values. However, as the particular μ value lies in the lower bound of the expected range of interface friction coefficients [21], eq. (23) will provide a conservative estimate, at least for preliminary design purposes.

The above findings support the validity of the Randolph *et al.* [1] conceptual model for the description of the axial pipeline friction response. Nevertheless, there is still an issue that needs to be resolved for the proper calibration of its parameters: Generation of excess pore pressures in their model is assumed to take place within a shear band of limited thickness of the order of $D/30$ [1], and their distribution below that zone is assumed to be parabolic. Note that this does not imply that the thickness of the shear band is a function of the pipeline diameter, but rather provide its order of magnitude in relation to the diameter of typical pipes. To shed some light on that assumption, we use the CFSFE model to obtain the actual 3-D spatial distribution of steady-state excess pore pressure beneath a pre-embedded pipeline, for different values of walking velocity (Figure 14). To calculate the shear-induced excess pore pressure, the pore pressure prior to beginning of axial movement is deducted from pore pressure at any arbitrary time during shearing. Although the magnitude of the excess pore pressure is of course different as we move from drained (Figure 14a) to partially-drained (Figure 14b) and undrained (Figure 14c) conditions, the distribution of excess pore pressure development remains practically unaltered; a finding that is compatible with Randolph *et al.* [1] hypothesis. This distribution is better depicted in Figure 15, where we plot the variation of excess pore pressures in the soil below the pipeline along cross-section A-A of Figure 14b.

Excess pore pressures in Figure 15 are normalized against the mean contact stress (q_b) and is presented for different consolidation stages, following the shearing stage.

Figure 14. Contours of excess pore pressure under the pipeline: (a) $vD/c_v \leq 6.6$; (b) $vD/c_v = 660$; (c) $vD/c_v = 6600$ ($E_R = 0.4$).

Figure 15. Distribution of normalised excess pore pressure beneath the pipeline for (a) coarse and (b) fine mesh (partially-drained loading $\nu D/c_v=660$, embedment ratio $E_R=0.4$).

Interestingly, the distribution of excess pore pressures is not uniform below the pipeline. It appears that there is a zone of thickness about $0.2D$, below which excess pore pressure distribution is indeed parabolic, as proposed by Schofield and Wroth [22] for excess pore pressure distribution in the consolidometer problem, an assumption adopted by Randolph *et al.* [1] for the distribution of excess pore pressures below the shear band. Of course the CFSFE model cannot simulate the formation of a shear band, but the fact that this zone appears to be independent of the mesh size indicates that shearing may take place at a more extensive plastic zone, rather than a thin shear band. Further elaboration on this issue is required, exactly due to the limitations of this numerical method.

4. SUMMARY AND PRACTICAL REMARKS

A detailed numerical study was presented in this paper, focusing on the elements contributing to vertical penetration and axial shearing resistance of partially embedded offshore pipelines. The main purpose of this study was two-fold: a) To provide the framework for effective stress analyses of similar problems using a critical state model, compare quantitatively and qualitatively the results against experimental measurements and other solutions, and highlight the key parameters governing the development of resistance to vertical and axial pipeline movements; b) To provide practical conclusions and simplified expressions for the analysis of offshore pipelines, while outlining their application limitations. Some practical conclusions are summarised below.

- Heave develops around the pipeline under fully undrained embedment conditions only; its beneficial contribution to the axial resistance must be carefully taken into account in

practice, and only in cases where the construction method will ensure undrained vertical penetration of the pipeline. A chart (similar to Figure 6 which applies to $E_R=0.45$ only) can be used to define the bounds between drained, partially-drained and undrained response, at least for preliminary estimates.

- Simplified formulas are proposed for the estimation of the contact enhancement factor ζ , both for wished-in-place (WIP) and pushed-in-place (PIP) pipelines. Heave development around PIP pipelines increases the contact enhancement factor by about 20% compared to WIP pipelines, with the effect being more prominent for “smooth” pipelines and higher embedment ratios.
- Axial walking response, in terms of the maximum value of axial resistance and its evolution with time, chiefly depends on the soil-pipeline interface drainage condition during shearing. For relatively high velocities, resulting in undrained and partially drained axial shearing, lower residual axial resistance is expected, a finding that is in line with experimental data [2]. In the light of this, a quantitative extension of the Randolph *et al.* [1] model is presented, accounting for the effects of the embedment ratio and the soil parameters. The formula for estimating the maximum equivalent friction factor F/W' as a function of the normalised time T^* (i.e. the shearing velocity) is calibrated for different sets of soil parameters, which appear to have an effect on the range of the partially-drained zone. The shape of the backbone curve providing the maximum equivalent friction factor appears to be insensitive to the embedment ratio of the pipeline, and a simplified expression is proposed to extent the use of the Randolph *et al.* [1] for varying embedment ratios.
- The interface friction coefficient μ has an unquestionable effect on the ultimate values of the equivalent friction factor. We therefore propose, at least for preliminary design purposes, to use the expressions derived for a lower-bound value $\mu=0.3$, and will thus provide conservative estimates of the axial resistance developing at the soil-pipeline interface.

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REFERENCES

- [1] Randolph MF, White DJ, Yan Y. Modelling the axial soil resistance on deep-water pipelines. *Géotechnique*. 2012;62(9): 837–46.
- [2] White DJ, Ganesan SA, Bolton MD, Bruton DAS, Ballard J-C, Langford T. SAFEBUCK JIP – Observations of axial pipe-soil interaction from testing on soft natural clays. Offshore Technology Conference. Houston2011. p. Paper OTC 21249.
- [3] Aubeny C, Shi H, Murff J. Collapse Loads for a Cylinder Embedded in Trench in Cohesive Soil. *International Journal of Geomechanics*. 2005;5(4):320-5.
- [4] Murff JD, Wagner DA, Randolph MF. Pipe penetration in cohesive soil. *Géotechnique*. 1989;39(2):213-29.
- [5] Merifield RS, White DJ, Randolph MF. The ultimate undrained resistance of partially embedded pipelines. *Géotechnique*. 2008a;58(6):461-70.

- [6] Merifield RS, White DJ, Randolph MF. The effect of pipe–soil interface conditions on undrained breakout resistance of partially-embedded pipelines. 12th Int Conf on Advances in Computer Methods and Analysis in Geomechanics. Goa2008b. p. 4249–56.
- [7] Merifield RS, White DJ, Randolph MF. The effect of surface heave on the response of partially-embedded pipelines on clay. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE. 2009;135(6):819–29.
- [8] Martin CM, White DJ. Limit analysis of the undrained bearing capacity of offshore pipelines. *Géotechnique*. 2012;62(8):647–63.
- [9] Krost K, Gourvenec S, White DJ. Consolidation around partially embedded seabed pipelines. *Géotechnique*. 2011;61(2):167–73.
- [10] Chatterjee S, Yan Y, Randolph MF, White DJ. Elastoplastic consolidation beneath shallowly embedded offshore pipelines. *Géotechnique Letters*. 2012;2(2):73–9.
- [11] Vermeer PA, Verruijt A. An accuracy condition for consolidation by finite elements. *International Journal of Numerical and Analytical Methods in Geomechanics*. 1981;5(1):1–14.
- [12] Dingle HRC, White DJ, Gaudin C. Mechanisms of pipe embedment and lateral breakout on soft clay. *Canadian Geotechnical Journal*. 2008;45(5):636–52.
- [13] Wroth CP. The interpretation of in situ soil tests. *Géotechnique*. 1984;34(4):449–89.
- [14] Chang MF, Teh CI, Cao L. Critical state strength parameters of saturated clays from the modified Cam clay model. *Canadian Geotechnical Journal*. 1999;36(5):876–90.
- [15] Zhou H, White DJ, Randolph MF. Physical and numerical simulation of shallow penetration of a cylindrical object into soft clay. *GeoCongress*. New Orleans: ASCE Special Geotech. Publication, 2008. p. 108–17.
- [16] Martin CM, Randolph MF. Upper bound analysis of lateral pile capacity in cohesive soil. *Géotechnique*. 2006;56(2):141–5.
- [17] Silva MF, White DJ, Bolton MD. An analytical study of the effect of penetration rate on piezocone tests in clay. *International Journal for Numerical and Analytical Methods in Geomechanics*. 2006;30(6):501–27.
- [18] Kim K, Prezzi M, Salgado R, Lee W. Effect of penetration rate on cone penetration resistance in saturated clayey soils. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE. 2008;134(8):1142–53.
- [19] Gourvenec SM, White DJ. Elastic solutions for consolidation around seabed pipelines. *Offshore Technology Conference*. Houston2010. p. Paper OTC 20554.
- [20] White DJ, Randolph MF. Seabed characterisation and models for pipeline-soil interaction. *International Journal of Offshore and Polar Engineering*. 2007;17(3):193–204.
- [21] DNV (Det Norske Veritas). Design of HP/HT pipelines for global buckling. RP-F110. Oslo, Norway2007.
- [22] Schofield AN, Wroth CP. Critical state soil mechanics. London, UK: McGraw-Hill, 1968.

Figure Captions:

Figure 1. Finite element mesh and geometry of the (a) two-dimensional plane-strain model, (b) three-dimensional model.

Figure 2. Schematic diagram of PIP pipeline installation resulting in surface heave.

Figure 3. Comparison of resistance-displacement curves for (a, b) PIP and (c) WIP pipelines.

Figure 4. Comparison of semi-analytical (Merifield et al. [7]) and numerical predictions of the (a) normalized contact width and (b) normalized contact perimeter along the soil-pipeline interface, with the experimental results of Dingle et al. [12].

Figure 5. Comparison of numerical and experimental [12] soil heave profiles.

Figure 6. Vertical pipeline resistance as a function of drainage conditions.

Figure 7. Numerical surface heave predictions for (a) undrained, (b) partially-drained, and (c) fully drained penetration (embedment ratio $E_R=0.45$).

Figure 8. Distribution of (a) total contact stresses and (b) excess pore pressure along soil-pipeline interface (results for normalized velocity $vD/c_v=8.33$).

Figure 9. (a) Contact stress components at the pipeline-soil interface and (b) contact effective stress derivation using the CFSFE model results ($E_R=0.5$).

Figure 10. Predictions of contact enhancement factor (a) comparison with previous studies for the case of a smooth WIP pipe, and (b) effect of installation method and surface roughness.

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Figure 15. Distribution of normalised excess pore pressure beneath the pipeline for (a) coarse and (b) fine mesh (partially-drained loading $vD/c_v=660$, embedment ratio $E_R=0.4$).