Finite Element Modeling of Uplift Pipeline-Soil Interaction in Dense Sand

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ABSTRACT
Buried pipelines are one of the most efficient and popular methods to transport natural gas and petroleum products. Geohazards and the associated ground movement represent a significant threat to pipeline integrity that may result in pipeline damage and potential failure. Pipelines are often buried at shallow depth and therefore the behaviour of soil at low stress level needs to be considered for proper modeling of the pipeline response when subjected to upward movement. In this study, finite element (FE) modeling of pipeline-soil interaction is presented, where the stress-stain behaviour of soil at low stress level, including post-peak softening, is implemented. At first, triaxial test results are simulated to validate the proposed model and numerical techniques. Pipeline-soil interaction in the plane strain condition is then simulated for uplift loading. The Arbitrary Lagrangian-Eulerian (ALE) method available in Abaqus/Explicit is used for FE modeling. One of the main advantages of this method is that it can simulate large deformation behaviour. The variation of non-dimensional uplift force with non-dimensional displacement is examined for different depths of embedment.

RÉSUMÉ
Canalisations enterrées sont l'une des méthodes les plus efficaces et les plus populaires pour le transport de gaz naturel et de produits pétroliers . Aléas géologiques et les mouvements au sol associée représentent une menace importante pour l'intégrité du pipeline qui peut entraîner des dommages causés au pipeline et l'échec potentiel . Les pipelines sont souvent enterrés à faible profondeur et donc le comportement de sol à faible niveau de stress doit être pris en considération pour une bonne modélisation de la réponse du pipeline lorsqu'il est soumis à un mouvement vers le haut . Dans cette étude , éléments finis (EF) la modélisation de l'interaction pipeline - sol est présentée , où le comportement contrainte - tache de sol à faible niveau de stress , y compris post-pic ramollissement , est mis en œuvre . Dans un premier temps , les résultats des tests triaxiaux sont simulées pour valider le modèle proposé et les techniques numériques . Interaction pipeline - sol à l'état de déformation plane est ensuite simulé pour le soulèvement de chargement . La méthode disponible dans Abaqus / Explicit arbitraire Lagrange - Eulerian (ALE) est utilisé pour la modélisation FE . L'un des principaux avantages de cette méthode est qu'elle permet de simuler le comportement de déformation importante . La variation de la force de soulèvement non - dimensionnelle avec un déplacement non - dimensionnelle est examinée pour différentes profondeurs de l'encastrement.

1 INTRODUCTION

Energy pipelines are one of the most efficient and popular ways to deliver natural gas and petroleum products from field development areas to market. The liquid hydrocarbon and natural gas products are usually transported through buried pipelines, which traverse large distances through a variety of soils. Geohazards and the associated ground movement represent a significant threat to pipeline integrity that may result in pipeline damage and potential failure. In certain situations, pipelines can be exposed to potential ground failures such as surface faulting, liquefaction-induced soil movements, and landslide induced permanent ground deformation (PGD). These ground movements might cause excessive stresses in pipelines and pipelines might be damaged. Therefore, both pipeline integrity and safety are major concerns for pipeline operators and agencies.

Theoretical and experimental studies were conducted in the past to determine the forces on pipelines or anchor plates for upward movement, namely Trautmann, 1983; Dickin, 1988; Schaminee et al., 1990; Ng and springman, 1994; Hsu and Liao, 1997; Hsu and Liao, 1998; Bransby et al., 2001; White et al., 2001; Palmer et al., 2003; El-Gharbawy, 2006; Chin et al., 2006; Schupp et al., 2006; Byrne et al., 2008; Cheuk et al., 2008; Wang et al., 2009; Chen and Chu, 2010; Chou et al., 2011; Chen et al., 2012; Kumar and Naskar, 2012; Horikawa et al., 2012; Shinkai et al., 2012; Williams et al., 2013, Chakraborty and Kumar, 2013; Jung et al., 2013. Schaminee et al. (1990) identified that for uplift loading, dilatant soil such as dense sand shows a stiff initial response up to the peak resistance which is followed by post-peak softening. Sherif (2006) conducted several model tests for uplift movement of pipe to investigate the response of pipeline buried in loose silty sand. Cheuk et al. (2008) presented a set of model test results for uplift resistance. In these tests a novel image analysis technique based on particle image velocimetry (PIV) and close range photogrammetry were used to track the soil movement. Based on these results, four stages of soil deformation mechanisms are proposed. In order to understand the mechanism further, FE analyses in the Lagrangian framework have also been performed (e.g. Yimsiri et al., 2004; Daiyan et al., 2011; Xie, 2012; Jung et al., 2013). Yimsiri et al. (2004) conducted a comprehensive FE analysis using Abaqus/Standard FE software with the Mohr-Coulomb and Nor-Sand soil constitutive models. The degradation of soil strength parameters after the peak was not considered in that study.

Pipelines are often buried at shallow depth and therefore the stresses in the soil around the pipe before any movement are generally lower than typical geotechnical problems such as foundations. Therefore, the behaviour of soil masses around the pipeline at low stress level needs to be considered.

The main focus of the present study is to simulate the response of buried pipelines in dense sand. Although limited, some experimental studies on dense sand at low stress level are available in the literature (e.g. Ponce and Bell, 1971; Stroud, 1971; Ahmed, 1973; Fukushima and Tatsuoka, 1984; Tatsuoka et al., 1986; Lancelot, 2006). Ponce and Bell (1971) showed that sand exhibits a strong increase in friction and dilatancy angles when the confining pressure decreases in triaxial tests. However, Fukushima and Tatsuoka (1984) found a weaker variation.

Another important experimental observation is that the behaviour of sand differs in triaxial and simple shear conditions. For example, Ahmed (1973) conducted tests on crushed silica sand in drained triaxial (TX) and plane strain (PS) loading conditions. The peak friction angle (ϕ'_p) obtained from his test results are shown in Fig. 1. Three key features of these test results need to be mentioned. Firstly, the peak friction angle for the plane strain condition (ϕ'^{PS}_p) is higher that the peak friction angle in the triaxial condition (ϕ'^{TX}_p), and the value of $\phi'^{PS}_p - \phi'^{TX}_p$ is higher at low stress level. Secondly, both ϕ'^{PS}_p and ϕ'^{TX}_p increase with increase in relative density. Finally, the peak friction angle decreases with increase in confining pressure.

The main objective of the present study is to analyze pipeline-soil interaction during uplift of buried pipes in dense sand. An advanced simulation tool suitable for large deformation analysis is used for the FE analyses. A modified Mohr-Coulomb (MC) model with confining pressure dependent peak friction angle and dilation angle is used. In addition, the dependency of mobilized friction angle (ϕ') and dilation angle (ψ) on engineering plastic shear strain (γ_p) is used to simulate strain hardening and softening behaviour for dense sand. The uplift resistance from the present FE analyses is compared with available experimental results.

2 MODELING OF SOIL BEHAVIOUR

The Mohr-Coulomb model is one of the simple models that reasonably represent the behaviour of sand. It has been used by many researchers in the past for pipeline-soil interaction analysis. In this study, a modified form of Mohr-Coulomb model is used incorporating the following key features as observed in laboratory tests.

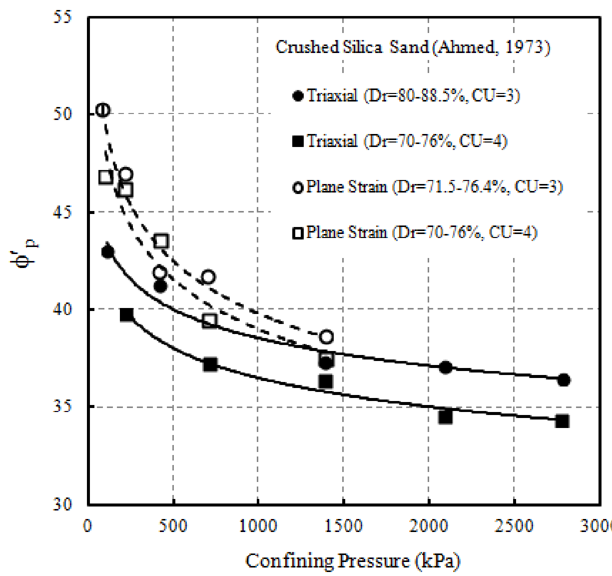

Figure 1. Test results for crushed silica sand (after Ahmed, 1973)

2.1 Angle of internal friction in PS conditions

Pipeline-soil interaction in plane strain condition is simulated in this study. The strength of sand is usually characterized by the angle of internal friction. Kulhawy and Mayne (1990) compiled a large volume of test data and showed that, in general, the peak friction angle of dense sand in PS is approximately 10% to 20% higher than the TX condition. Experimental results on dense sands also show that ϕ'^{PS}_p is more than 5° higher than ϕ'^{TX}_p (Schanz and Vermeer, 1996). Furthermore, experimental evidence shows that ϕ'_p decreases with increase in mean effective stress at failure (p'), and generally follows a linear relation with lnp'. Bolton (1986) analyzed the strength and dilatancy of 17 sands in TX and PS tests and proposed the following empirical relations:

$$\phi'^{TX}_p - \phi'^{TX}_c = 3I_R \qquad \text{for triaxial} \qquad [1]$$

$$\phi'^{PS}_p - \phi'^{PS}_c = 5I_R \qquad \text{for plane strain} \qquad [2]$$

Where I_R is the relative density index defined as $I_R = I_D (Q\text{-ln}p')\text{-}R$ with I_D=relative density (=D_r(%)/100). The subscripts p and c represent the peak and critical state, respectively. Bolton (1986) also showed that the values of Q=10 and R=1 fit most of the test data, although it might vary with type of sand and p' (Chakrabarty and Salgado 2010). As triaxial tests are widely used for geotechnical characterization, appropriate care need to be taken for estimation of ϕ'_p for pipeline-soil interaction analysis in plane strain condition. It is to be noted here that a similar attempt has been taken to estimate ϕ'^{PS}_p from direct shear test results (Lings and Dietz, 2004) and showed that $\phi_p^{\prime PS}$ is approximately 5 degrees higher than the peak friction angle obtained from direct shear test.

Equation 2 is used to model pipeline-soil interaction in PS condition in the present study, although the authors understand that additional laboratory tests at low p' are required to check the validity of this equation further.

Unlike ϕ'_p, the critical state friction angles may not differ considerably in PS and TX conditions. Experimental evidence shows that $\phi_c^{\prime PS}$ is few degrees higher than $\phi_c^{\prime TX}$. Bishop (1961) and Conforth (1964) conducted drained tests on sands over a range of densities at a wide range of confining pressure and showed that $\phi_c^{\prime PS}$ is approximately 4° higher than $\phi_c^{\prime TX}$. Similar results were obtained from laboratory tests on Toyoura sand (Tatsuoka et al., 1986; Pradhan et al., 1988), and have shown that $\phi_c^{\prime PS} \approx 34.5° - 38°$ while $\phi_c^{\prime TX} \approx 33°$.

The maximum dilation angle (ψ_p), which occurs at the peak shear strength, are related to the peak and critical state friction angles in plane strain condition as (Bolton 1986):

$$\phi_p^{\prime PS} = \phi_c^{\prime PS} + 0.8\psi_p \qquad [3]$$

In this study, $\phi_c^{\prime TX} = 31°$ and $\phi_c^{\prime PS} = 35°$ are used.

2.2 Stress-strain behaviour of dense sand

In the modified Mohr-Coulomb model, the mobilized shear strength parameters (ϕ' and ψ) are varied with accumulated plastic shear strain (γ_p) as shown in Fig. 2. In the pre-yield zone, both ϕ' and ψ increase from (ϕ'_{in} and ψ_{in}) to the peak values at γ_p^p, and therefore strain hardening occurs in this zone.

Experimental evidence shows that the plastic strain at peak, γ_p^p decreases with increasing relative density and increases with increasing p'. For example, from direct shear tests, Lings and Dietz (2004) showed that for a dense sand (D_r=90%) the peak friction angle is mobilized at horizontal displacement of 1.5 mm and 3.5 mm under normal stress of 25 kPa and 251 kPa, respectively. In order to capture the non-uniqueness of γ_p^p, in this study the behaviour is defined as:

$$\gamma_p^p = \gamma_p^c \left(p'/p'_a\right)^{0.252} \qquad [4]$$

$$\gamma_c^p = \left(22.1 - 11.2D_r\right)/100 \qquad [5]$$

where γ_c^p = strain softening parameter, which is explained further in the following sections, and p'_a= reference pressure = 100 kPa.

The following sine function is then used to model the variation of mobilized ϕ' and ψ in the pre-yield zone.

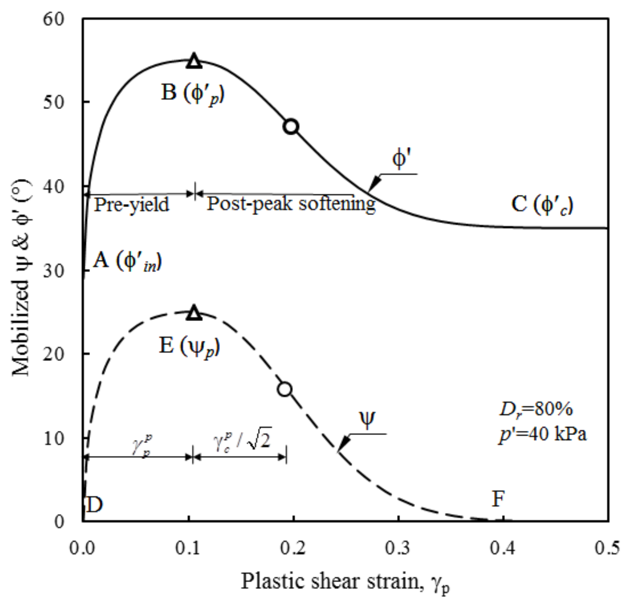

Figure 2. Modeling of stress-strain behaviour of dense sand

$$\phi' = \phi'_{in} + \sin^{-1}\left(\frac{2\sqrt{\gamma^p \gamma_p^p}}{\gamma^p + \gamma_p^p}\right)\sin\left(\phi'_p - \phi'_{in}\right) \qquad [6]$$

$$\psi = \sin^{-1}\left(\frac{2\sqrt{\gamma^p \gamma_p^p}}{\gamma^p + \gamma_p^p}\right)\sin\left(\psi_p\right) \qquad [7]$$

The value of ψ_p can be calculated using Eq. 3. The lines AB and DE in Fig. 3 show the variation of ϕ' and ψ, respectively, in the pre-yield zone for D_r=80% and p'=40 kPa.

If the shearing is continued, both ϕ' and ψ will decrease with plastic strain as shown in Fig. 2. This zone is referred as "post-peak softening zone." The following exponential functions are used to define the curve BC and EF to model the variation of ϕ' and ψ, respectively.

$$\phi' = \phi'_c + \left(\phi'_p - \phi'_c\right)\exp\left[-\left(\frac{\gamma^p - \gamma_p^p}{\gamma_c^p}\right)^2\right] \quad \text{curve BC} \quad [8]$$

$$\psi = \psi_p \exp\left[-\left(\frac{\gamma^p - \gamma_p^p}{\gamma_c^p}\right)^2\right] \quad \text{curve EF} \quad [9]$$

The strain softening parameter γ_c^p controls the shape of the post-peak curves. After some algebraic calculation, it can be shown from Eqs. (8) and (9) that the point of inflection of the post-peak softening curve occurs at a shear strain of $\gamma_c^p/\sqrt{2}$ greater than γ_p^p which is shown by the open circle in Fig. 2. It is to be noted here that the modified Mohr-Coulomb model with strain dependent ϕ'

and ψ have also been used in the past for modeling dense sand. Anastaspoulos et al. (2007) used a simple straight line to model post-peak degradation. Jung et al. (2013) used that concept for pipeline-soil interaction analysis. In those studies, pre-yield behaviour was not considered, rather the stress-strain behaviour before the peak was assumed to be elastic.

The soil constitutive model is then implemented in Abaqus/Explicit using a user subroutine written in FORTRAN.

3 PERFORMANCE OF SOIL CONSTITUTIVE MODEL

In order to show the performance of the soil constitutive model described in the previous sections and also to validate the present FE implementation, a set of triaxial test results (Hsu and Liao, 1998) are simulated first. The FE simulation is performed for consolidated isotropically drained triaxial tests on dense sand (D_r=70%) for a wide range of confining pressures of 20-320 kPa. The value of $\phi_c'^{TX} = 31°$ is used. The variation of $\phi_p'^{TX}$ is defined by using Eq. (1). The calculated deviatoric stress and volumetric strain are shown in Fig. 3, which show that the proposed soil constitutive model can successfully simulate the stress-strain behaviour of dense sand for a wide range of confining pressures including the low stress levels, which is the interest of the present study in pipeline-soil interaction modeling. These observations provide confidence in the modeling approach and numerical procedures implemented in Abaqus/Explicit FE analysis.

4 FINITE ELEMENT FORMULATION

Two-dimensional pipeline-soil interaction analyses are performed using the Arbitrary Lagrangian-Eulerian method available in Abaqus/Explicit 6.10 EF1. The main advantages of using Abaqus/Explicit over Abaqus/Standard is that the pipe can be moved sufficiently large distance avoiding numerical issues due to mesh distortion as encountered in the Abaqus/standard, especially in the zone of shear strain localization in the shear bands. Therefore, the formation of shear band can be better simulated in Abaqus/Explicit.

Figure 4 shows the typical FE model used in this study. Taking the advantage of symmetry, only half of the domain is modeled. The depth of the pipe is measured in terms of *H*/*D* ratio, where *H* is the depth from the top of the soil to the center of the pipe and *D* is the external diameter of the pipe. The centre of the pipe is placed at 2D above the bottom boundary. The thickness of soil above the centre of the pipe varies with *H*/*D* ratio. For example, in the simulation of *H*/*D*=4, the distance from the centre to the ground surface is 400 mm for D=100 mm. The left boundary is placed at 2.5 D from the pipe. The distances from the pipe to the bottom and left boundaries are sufficiently large and therefore boundary effects on predicted uplift resistance, displacement and soil failure mechanisms are not found. This is verified from a number of FE analyses, setting these boundaries at larger distances than that shown in Fig. 4.

For FE modeling of soil, 4-node bilinear plane strain quadrilateral, reduced integration, hourglass control element (CPE4R) is used. The pipe is modeled as a rigid body. Abaqus/cae is used to generate the finite element mesh. The structured mesh, as shown in Fig 4, is generated by zoning the soil domain. Denser mesh is used near the pipe. The total number of elements and shapes can be defined in the structured mesh, which cannot be done in the auto generated default meshing option in Abaqus. In this study, structured mesh is used because it gives better results, less numerical issues and computationally efficient than with auto generated mesh.

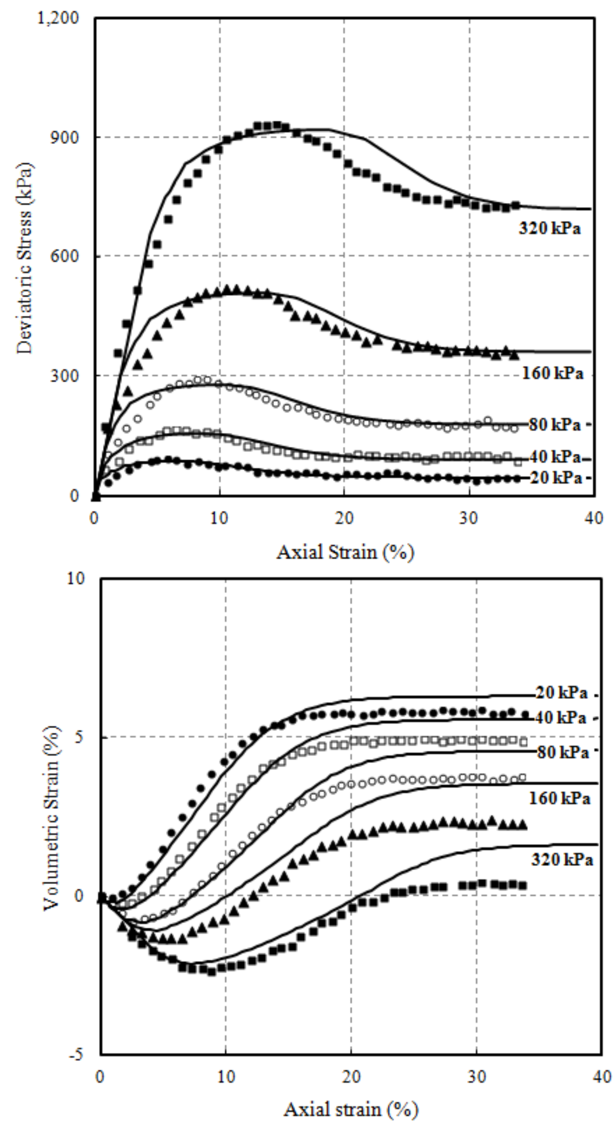

Figure 3. Comparison between FE and laboratory test results of Hsu and Liao (1998)

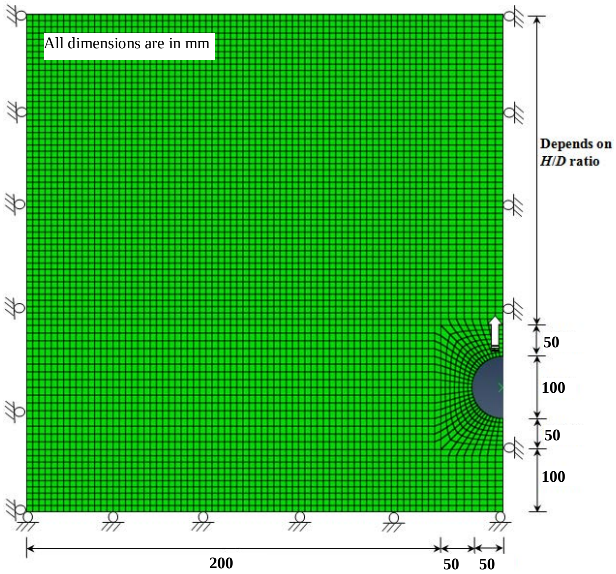

Figure 4. Typical finite element mesh

The bottom of the FE domain is restrained from any vertical movement, while all the vertical faces are restrained from any lateral movement using roller supports (Fig. 4). No displacement boundary condition is applied on the top face, and therefore the soil can move freely.

The interface between pipe and soil is simulated using the contact surface approach available in Abaqus/Explicit. The Coulomb friction model is used for the frictional interface between the outer surface of the pipe and sand. In this method, the friction coefficient (μ) is defined as $\mu=\tan(\phi_\mu)$, where ϕ_μ is the pipeline-soil interface friction angle. The value of ϕ_μ depends on the interface characteristics and relative movement between the pipe and soil. The larger value of ϕ_μ represents the characteristics of rough uncoated pipes with rusty or corroded surfaces, while the lower values would correspond to pipes with smooth coating. The value of ϕ_μ varies between $\phi_p^{\prime TX}$ and $\phi_p^{\prime TX}/2$ (Yimsiri et al, 2004). The value of μ equal to 0.32 is used in this study.

The numerical analysis is conducted in two main steps. The first step is a geostatic stress step that accounts for the effects of soil weight and defines the initial stress state in the soil. The initial stress or the geostatic stress step definition is very important for pipeline-soil interaction analyses. It is to be noted here that if the geostatic condition is not properly modeled with appropriate initial stress condition, the response in subsequent loading might be erroneous and/or additional numerical issues might be encountered, because the behavior of sand is effective stress dependent. In this study, it has been properly defined and the calculated stresses at the end of geostatic step are same as expected in situ stress.

In the second step, the pipe is moved in the upward direction specifying a displacement boundary condition at the reference point of the pipe.

5 SIMULATION OF PIPELINE-SOIL INTERACTION

After verification of soil constitutive model performance in triaxial condition, FE simulations are performed for pipelines buried in dense sand (D_r=80%) under uplift loading in plane strain condition. The FE results are first verified with the results of model tests conducted by Trautmann (1983). These test results have also been used by previous researchers to validate numerical modeling performance. For example, Yimsiri et al. (2004) reanalyzed the direct shear test results presented by Trautmann and O'Rourke (1983) for estimation of soil parameters and used $\phi_c' = 31°$ in their FE analyses. As mentioned before that ϕ' in PS is higher than ϕ' in triaxial and direct shear test (Pradhan et al., 1988; Lings and Dietz, 2004) a value of $\phi_c' = 35°$ is used in the present study. The peak friction angle is calculated using Eq. (2) with a maximum value of $\phi_p'^{PS} - \phi_c'^{PS}$ equal to 20° as suggested by Bolton (1986). The unit weight of dry sand used for model test was 17.7 kN/m^3 that corresponds to a relative density of 80%. A Poisson's ratio of 0.2 is used, which is considered as the best representative value for dense sand (Jefferies and Been, 2006). The modulus of elasticity (E) is varied with initial mean effective stress (σ_m) as $E = E_0\left(\sigma_m / \sigma_{m(ref)}\right)^n$, where E_0 is the value of E at reference pressure ($\sigma_{m(\mathrm{ref})}$), and n is a material constant. Parameters used in the FE analysis are summarized in Table 1.

Table 1: Soil Parameters used in the FE analysis

Parameter	Values
External diameter of pipe, D	100 mm
Poisson's ratio, ν_{pipe}	0.3
E_0	15,000 kN/m^2
n	0.5
$\sigma_{m(\mathrm{ref})}$	100 kN/m^2
Poisson's ratio, ν_{soil}	0.2
Critical state friction angle, ϕ'_c	35°
Unit weight, γ	17.7 kN/m^3
Interface friction co-efficient, μ	0.32
Depth of pipe, H/D	1.5, 4 & 8

6 RESULTS

The solid lines in Fig. 5 show the variation of dimensionless force ($F/\gamma HD$) with dimensionless upward displacement (v/D) from the initial position for three burial depths (H/D=1.5, 4 and *8*). As shown, the force on the pipe increases with displacement and reaches to the peak and then decreases in the post-peak zone. In order to show the performance of the present FE model, the force-displacement curves obtained in the full-scale tests (Trautmann, 1983) are also plotted in Fig. 5. The present FE model can successfully simulate the trend of force-displacement curves, although in the FE analyses for lower H/D (=1.5 & 4) the peak resistance is developed at larger displacement and the rate of post-peak softening is slower than that observed in the full-scale tests. In the present FE analyses, the exact conditions of the tests, including soil properties, may not be properly simulated. Moreover, the soil around the pipe is relatively at very low stress level because these tests were conducted at shallow depths. As mentioned before, the modeling of stress-strain behaviour of soil at such low stress level is difficult. These might be some causes of discrepancy between the force-displacement curves obtained from the full-scale tests and FE analyses.

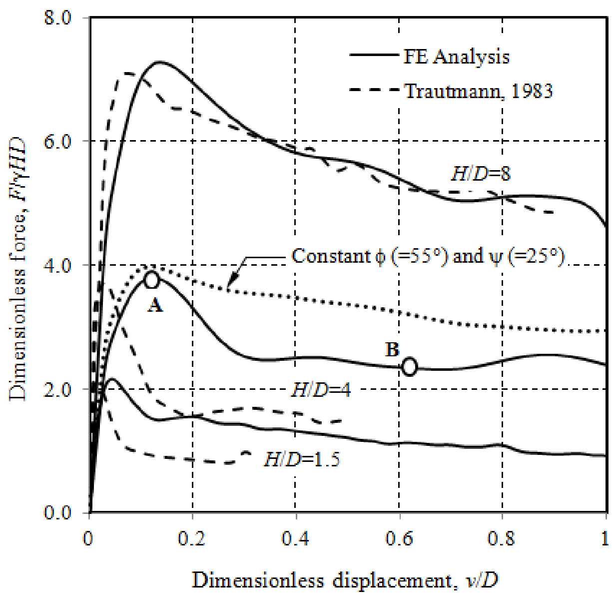

Figure 5. Comparison of FE results with the large scale test results (Trautmann, 1983)

One of the key questions is whether the post-peak softening behavior of soil, as shown in Fig. 2, is important for modeling uplift behavior of pipeline. To show that, FE simulation is performed for constant values of ϕ' (=55°) and ψ (=25°) for *H*/*D*=4. The force-displacement curve is shown in Fig. 5. Note that, these values should be carefully selected that should be representative of average values of ϕ' and ψ although they actually vary with strain. In general, the values of ϕ' and ψ should be lower than the peak and higher than critical state. Number of previous studies simulated the response using such constant values. As shown in Fig. 5 that constant ϕ' and ψ cannot simulate the force-displacement curves properly, especially the post-peak zone. There is a slight decrease in uplift force after the peak because the burial depth is reduced with upward movement of the pipe. However, it is significantly different from the observed softening in the full-scale tests. Therefore, the post-peak stress-strain behaviour of soil needs to be incorporated in the FE analyses for better simulation.

Figure 6 shows the mobilized ϕ' and ψ for *H*/*D*=4 at a very large displacement (*v*/*D*=0.62). The point B in Fig. 5 shows the location. As shown in Fig. 6, ϕ' and ψ mainly vary in a wedge of soil above the pipe where plastic shear strain is developed. Outside this wedge the soil is elastic. The soil elements around the shear band (shown by dashed line) reach to the critical state ($\phi'=\phi'_c$ and ψ=0) because of significant plastic shear strain. Therefore, the soil block right side of this band mainly moves upward due to upward movement of the pipe. Not only in the shear band, a zone of soil above the pipe is also reached to the critical state. Therefore, the soil moves easily into the void under the pipe at this stage.

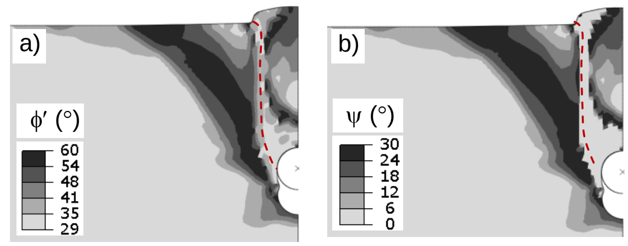

Figure 6. Contour plot of ϕ' and ψ for *H*/*D*=4 and *D*=100mm at v/D=0.62

7 CONCLUSIONS

The pipeline-soil interactions associated with relative movement of the pipeline in the vertical upward direction is numerically investigated in this study. The FE simulations are performed for two-dimensional plane strain condition. The key features considered in modeling of the behaviour of dense sands are: (i) the decrease of peak friction angle with increase in effective stress at failure, (ii) an improved stress-strain behaviour of dense sand, including the pre-yield hardening and post-peak softening with plastic shear strain; and (iii) plane strain strength parameters, which are different from triaxial or direct shear strength parameters. The FE modeling is performed using Abaqus/Explicit FE software, which can simulate even large strain response utilizing adaptive meshing techniques.

The present FE model can simulate successfully the triaxial test results for a wide range of confining pressures, including the tests under low confining pressures.

For pipelines, the calculated force-displacement curves match well with model test results of Trautmann (1983). The peak dimensionless force increases with increase in burial depth ratio (*H*/*D*). The results obtained from the present FE analyses are consistent with previous studies.

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