Uplift Resistance of Buried Pipelines in Dry and Unsaturated Sands: Comparison of Analytical and FE Model Results with Large-Scale Test Data

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Abstract: Pipelines are commonly buried underground to provide environmental stability, temperature insulation and mechanical protection. These pipelines are frequently subjected to earthquake induced upward displacements, which can cause significant social-economic loss to consumers and utility management. Further, high thermal and pressure of the conveying medium can induce differential stresses on the axial restrained pipe to result upward buckling of the pipeline that can disturb the serviceability conditions. The uplift resistance from soil cover protects the pipe against such unwanted movements, representing it as a vital design parameter, in that pipeline integrity under operating conditions relies on its value. The paper presents full-scale uplift results and finite-element parametric studies conducted to investigate the effects of dimensionless cover heights (soil cover height to diameter ratio), soil relative density and moisture content on the peak uplift resistance of pipes. The results showed that the available analytical models could predict realistic peak uplift resistance for pipes buried at shallower depths, however, they can substantially under-predict the pipe loads/uplift resistance especially when buried at deeper embedded depths and non-dry soil conditions. The results of the current study are useful for pipe designs against earthquakes and/or severe operating conditions induced uplift displacements in sandy soils.

Keywords: Pipelines, Earthquakes, Upward buckling, Uplift resistance, Non-dry soil, Analytical models.

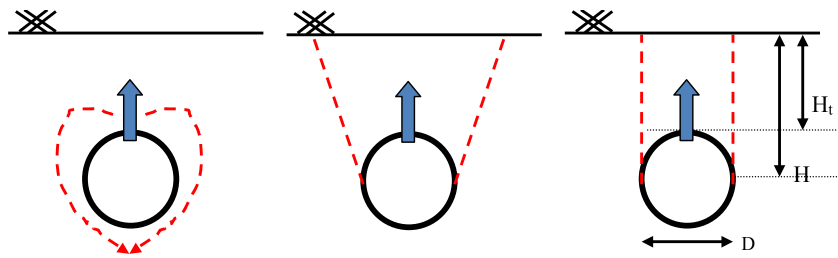
INTRODUCTION

Pipelines are commonly buried underground to provide environmental stability, temperature insulation and mechanical protection. These pipelines are frequently subjected to earthquake induced upward displacements, which can cause significant social-economic loss to consumers and utility management. Further, high thermal and pressure of the conveying medium can induce differential stresses on the axial restrained pipe to result upward buckling of the pipeline that can disturb the serviceability conditions. The uplift resistance from soil cover protects the pipe against such unwanted movements, representing it as a vital design parameter, in that pipeline integrity under operating conditions relies on its value.

Design of a buried pipeline requires a minimum depth of soil cover to provide the sufficient uplift resistance against upward displacements. Due to the severe operating conditions of oil and gas transport lines, the depth of soil cover needs to be calculated on the basis of pipeline operating conditions and soil mobilization resistance. Further, burial depth requirement stands a significant portion of the total construction cost of the pipeline. Therefore, the proposal of soil cover height in high pressure and high temperature (HPHT) pipelines requires a compromising decision which provides sufficient soil cover height while minimizing the unwanted construction costs making the design economically viable. Thus, an accurate estimation of peak uplift resistance from soil cover is vital in the design phase of HTHP pipelines. The paper presents full-scale uplift results and finite-element parametric studies conducted to investigate the effects of dimensionless cover heights (soil cover height to diameter ratio), soil relative density and moisture content on the peak uplift resistance of pipes subjected to vertical displacement in coarse-grained soils. Full scale experimental results were compared with the FE results for validation. The results from the current study are compared to ASCE design guidelines as well as available analytical models which predict the uplift resistance of soil.

LITERATURE REVIEW

Substantial analytical and numerical works have been conducted by previous researchers to investigate the uplift resistance and failure mechanisms of soil during upward displacement of pipes (Trautmann et al, 1985; Ng and Springman 1994; Bransby et al, 2001; White et al, 2001; Vanden Berghe et al, 2005; Chin et al, 2006; Schupp et al, 2006; Cheuk et al, 2008). Previous observations from model tests suggest that the uplift mechanism (i.e. inclination of the shear zone) depends on the initial state of the sands. A localized shear with a flow-around mechanism was observed in model tests conducted by Bransby et al. (2001) for very loose sands (Fig. 1c). A similar mechanism was also observed by White et al. (2001) for initially dense sand after the peak resistance is achieved. Such mechanism has also been numerically predicted by Vanden Berghe et al. (2005) for very loose sand. For medium to dense soil, inclined slip surface model (Fig. 1b) was experimentally proven to be a closer approximation for the real deformation mechanisms (Thusyanthan et al, 2010). Cheuk et al. (2008) showed that the average inclination of the shear zones is influenced by the soil density, with denser soil being more dilatant.



(c) Flow around model (b) Inclined slip surface model (a) Vertical slip surface model

Fig. 1. Different uplift mechanisms of buried pipes in granular soils

Several prediction models have been reported in literature to assess the peak uplift resistance of pipes buried in granular soils. Schaminée et al. (1990) have proposed a limit equilibrium solution (known as vertical slip model; Fig. 1c) to estimate the uplift resistance (Eq. 1) due to shear resistance along the vertical slip surface and weight of the soil block. Vermeer and Sutjiadi (1985) described a solution (Eq. 2) with straight shear bands extending to the soil surface considering the normality condition [i.e. friction angle (ϕ) = Dilation angle (ψ)]. Ng and Springman (1994) also proposed a solution similar to vertical slip model for the case of earth pressure coefficient, K=1 (Eq. 3). White et al. (2001) proposed an alternative limit equilibrium solution (Eq. 4) which considers Bolton's flow rule (Bolton, 1986) to infer the angles of friction and dilation linked to relative density (γ' - effective unit weight of backfill soil) and stress levels.

$$P = \gamma' HD + \gamma' H^2 K \tan\phi \qquad \text{Eq. 1}$$

$$P = \gamma' HD + \gamma' H^2 K \tan\phi_{\max} \cos\phi_{crit} \qquad \text{Eq. 2}$$

$$P = \gamma' HD + \gamma' H^2 \tan\phi_{\max} \qquad \text{Eq. 3}$$

$$P = \gamma' HD + \gamma' H^2 \tan\psi + \gamma' H^2 (\tan\phi_{\max} - \tan\psi)\left[(1+K_o) - (1-K_o)\cos 2\psi / 2\right] \quad \text{Eq.4}$$

Cheuk et al. (2008) argued that vertical slip model does not capture the deformation at the peak resistance, while the solution based on inclined planes with normality (Eq.2) overestimates the dilation (thus width of soil block lifted). The limit equilibrium solutions which capture the realistic shear band inclination via flow rules (such as in Eq. 4) have been reported to predict accurate uplift resistance.

OVERVIEW OF THE FULL SCALE EXPERIMENTS

Full scale experiments were carried out using a steel pipeline of 3m in length and 200mm in diameter. A full scale test tank of 2250mm width, 2500mm height and 5000mm length was used in the experiment. The backfill cover was loose fine sand (similar to Fraction E sand) with bulk unit weight of 15kN/m^3 (relative density ~ 35%, Thusyanthan et al, 2010). Firstly, the pipeline, which was attached with load cells (capacity 20 tons) and vertical side bars with a tape measure, was lowered down to the full scale test tank using an overhead crane. Then, it was covered with layers of soil (each ~30cm in thickness) with uniform compaction of each layer using Rammers. Having finished the test box preparation, the pipe was raised using manual pulleys at the two ends of the pipe at uniform rate of ~10mm/min. The readings of the load cell were recorded by a digital camera, and the displacement readings were measured by survey teams using the tape measure on the vertical side bars. A series of uplift experiments were carried out at various cover heights. In this paper, results of only H_t/D ratios of 6 and 8 are presented for comparison with FE modeling. These tests were conducted at in-situ moisture content (approximately 5% moisture).

FE MODELING OF UPLIFT PIPE DISPLACEMENT

Overview

FE modelling of uplift pipe displacement was undertaken in ABAQUS with geometric non-linearity and large strain formulation. This ensures that the large strains induced in the elements around the pipe during uplift of the pipe are modeled correctly. The non-linear geometry option (NLGEOM) in ABAQUS considers the changes in geometry during the analysis, and thus the equilibrium is achieved using the current configuration (i.e. current nodal position) of the model. All the analyses were performed under plane strain conditions and the model uses soil and pipe elements with 8-noded biquadratic, plane strain, reduced integration (CPE8R) elements (ABAQUS, 2007). Figure 2 shows the geometry and the mesh discretisation of the FE model used to simulate the vertically loaded pipeline experiments. The wall boundaries were assumed to be smooth and supported only in the normal direction. The pipe was pulled vertically by imposing equal uplift displacement on all pipe nodes and was set to move freely in the lateral direction. Adaptive meshing has been incorporated in the analyses to control the mesh distortions that result from large deformations of the soil caused by upward pipe displacements. The behaviour of the pipe is assumed to be linear elastic, whereas non-linear elasticity is assumed for the behaviour of soil surrounding the pipe.

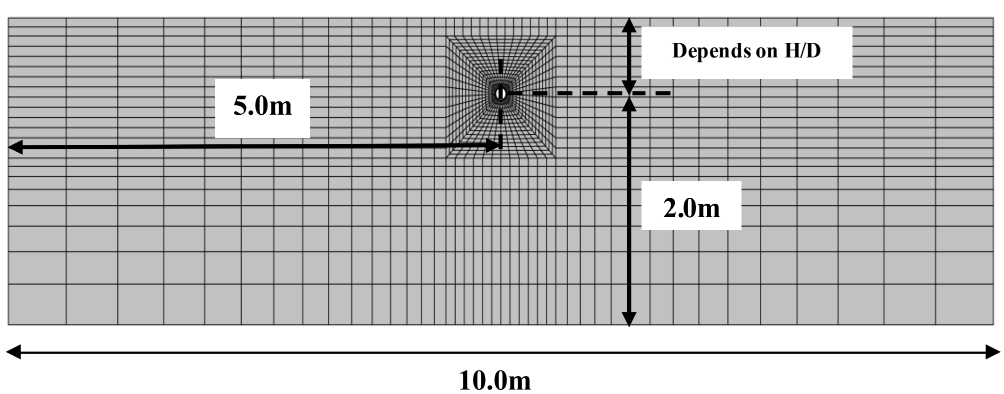


Fig. 2. Geometry and mesh discretisation of the model used for the FE analyses

The interaction between the pipe and soil has been modeled on the basis of Coulomb friction model which relates the maximum allowable frictional shear stress (τ_{crit}) across the interface to the contact pressure ($\sigma_n^{'}$) between the pipe and soil. The allowable frictional stress is given by $\mu\sigma_n^{'}$ where $\mu(\tan\phi_\mu)$ is the interface friction coefficient. The contacting surfaces will stick together and the behaviour remains elastic when $\tau < \tau_{crit}$. The slipping along the interface between the buried pipe and surrounding soil takes place once τ produced in the interface reaches τ_{crit}. This behaviour was modelled using the finite movement solution available in ABAQUS (ABAQUS, 2007). In the current study, ϕ_μ was set to equal to half of the peak frictional angle of soil (Cheong, 2006 & Yimsiri et al, 2004).

Constitutive Models

Pipe model

The pipe was assumed to have a Young's modulus of 204 GPa with a Poisson's ratio, of 0.3, for all the analyses conducted in this paper. Nevertheless, since the pipe was displaced as a rigid body, the pipe stiffness is essentially infinity, and thus, the soil stresses imposed on the pipe and the pipe deformations are negligible (Cheong, 2006).

Soil model

Soil behavior was modeled using Nor-Sand model. Nor-Sand model is a generalised Cambridge-Type constitutive model for sand, developed on the basis of the critical state theory. It uses the state parameter concept (Been & Jefferies, 1985) and attempts to accurately reproduce dilation and softening on the dry side of the critical state. This is achieved by postulating infinite isotropic normal consolidation loci (NCL), which allows a separation of the intrinsic state from the over-consolidation state. A main feature of Nor-Sand model is the use of rate-based hardening using the state parameter to size the yield surface. The original Nor-Sand model was proposed by Jefferies (1993) and was implemented into STANDARD finite elements by Dasari and Soga (2000). In order to enhance the model performance, three modifications were made by Cheong (2006). They include (i) a new definition for the critical state, (ii) lode angle dependency on the critical state parameter and (iii) the evolution of yield surface with respect to plastic shear strain. Nor-Sand code was implemented into explicit finite elements in order to be benefited by explicit simulations (Robert, 2010).

The calibration of the Nor-Sand model was performed on the basis of sand type (i.e. Chiba sand) having similar particle size gradation with sand used in large scale tests (Fig. 3). Chiba sand was previously calibrated by Robert (2010) for both saturated and unsaturated states through a series of triaxial laboratory tests.

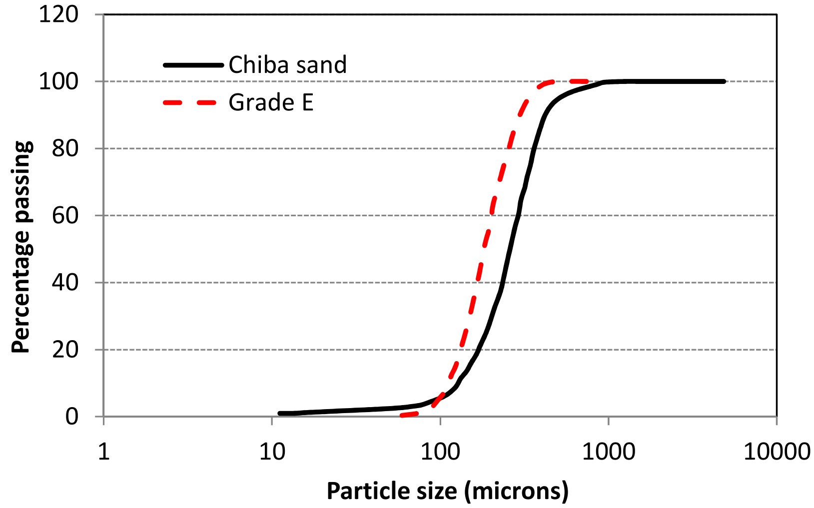


Fig. 3. Particle size distribution of sands used in the study

RESULTS

Simulating large scale tests

Analyses were performed to simulate the large scale tests using above modeling methodology. Simulations were performed on the basis of assuming the initial state of the sand as in dry condition. This is because the large scale tests were conducted at in-situ moisture content at which the water saturation of the sand (i.e. ~15%) is even lower than residual degree of saturation for Chiba sand. Figure 4 illustrates the soil moisture test data for Chiba sand obtained at dry density of 1.47g/cm^3 (~48% relative density). The initial dry density of the soil used in large scale tests is ~1.4g/cm^3.

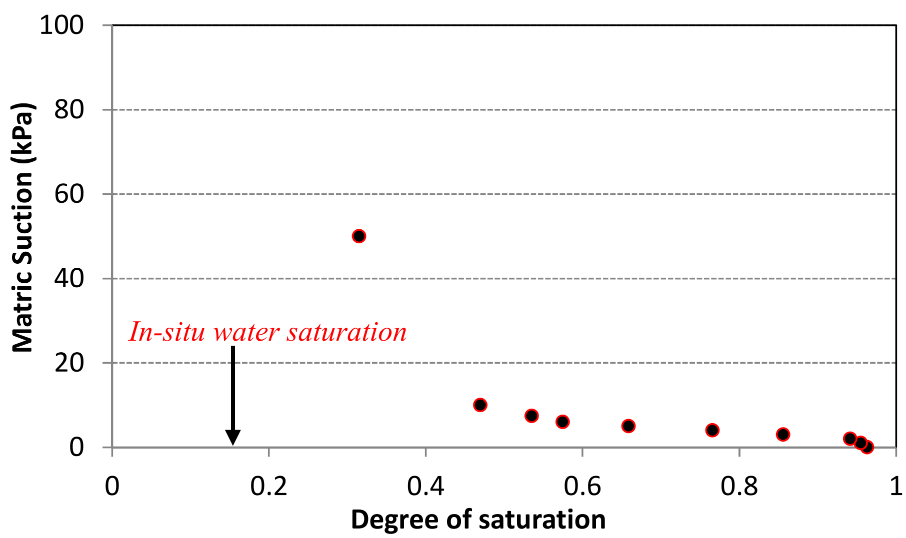


Fig. 4. Soil moisture test data at dry density of 1.47g/cm^3 (Robert, 2010)

Fig.5 shows the load (uplift resistance)-displacement plots obtained from the analyses in comparison with the FE results for H_t/D case 6 and 8. The deformation mechanisms of the FE model at ~0.5 dimensionless displacement (i.e. displacement/diameter) were showed in Fig. 6a&b (plastic deviatoric strains are shown). Larger pipe loading was resulted for deeply embedded pipes due to larger confinement effects from soil. i.e. The larger/broader shear band evolution induces higher pipe uplift resistance in $H_t/D=8$ case compared to smaller/narrower shearing of soil at $H_t/D=6$ (Fig.6).

FE model, which assumes dry sand behavior, predicts similar peak loading as compared to uplift resistance observed in large scale test for $H_t/D=8$, whereas the model under-predicts the experimental uplift resistance by ~8% for $H_t/D=6$. This slight under-prediction of peak uplift resistance at shallower depth could be due to the effects of suction and apparent cohesion of sand particles at the in-situ moisture content. The effect of suction and apparent cohesion diminishes (i.e. relatively low) at larger confining stresses (i.e. $H_t/D=8$) and hence the dry Nor-Sand model simulates similar peak uplift resistance when pipe is buried at deeper depths. The models predict stiffer response at lower pipe displacements in both H_t/D cases. This can be due to slightly coarseness (i.e. higher elastic stiffness) of the calibrated Chiba sand compared to grade E sand used in the large scale tests.

The prediction of the peak dimensionless uplift resistance ($F/\gamma HDL$, F – Peak uplift resistance, γ – Unit weight of soil, H – Soil cover height, L - Pipe length) from FE model was compared to the experimentally observed values and ASCE (1984) predictions in Fig. 7. It can be seen that the FE model prediction follows the ASCE guideline for pipe loading at $\phi'=36^o$ (ϕ' at 1.4g/cm^3 for Chiba sand is ~ 35^o). The slight increase in the peak dimensionless load from large scale test at $H_t/D=6$ could be due to the suction and apparent cohesion effect of sand at in-situ moisture condition.

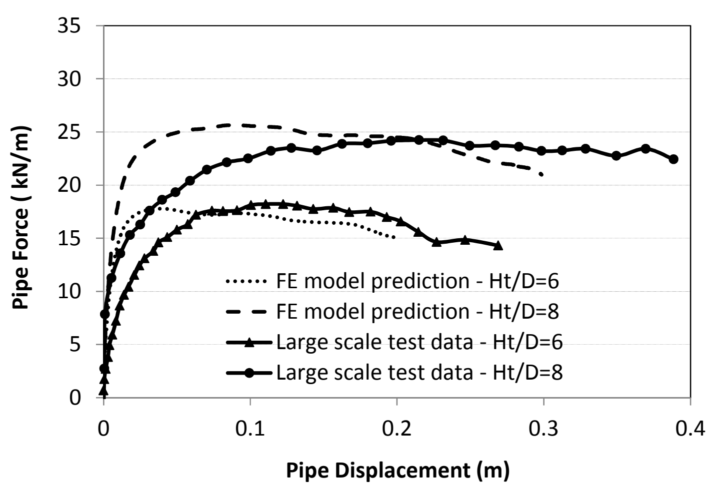
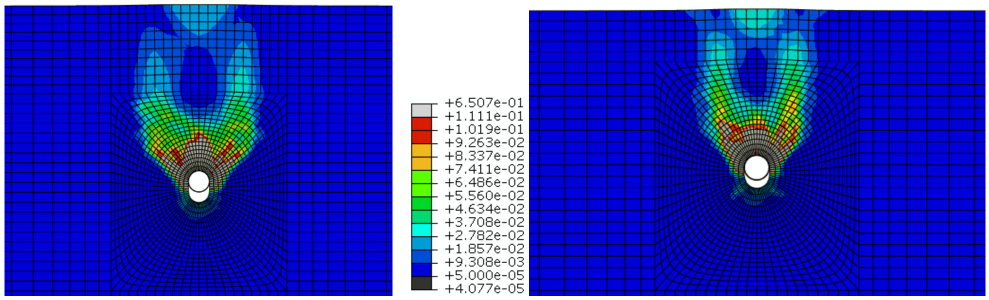


Fig. 5. FE model prediction of the pipe uplift resistance -displacement in comparison with test data



(a) H/D=6 (b) H/D=8

Fig. 6. Deformation mechanism of the FE model at (a) H/D=6 & (b) H/D=8 at peak pipe mobilization (Shear strain contours are shown).

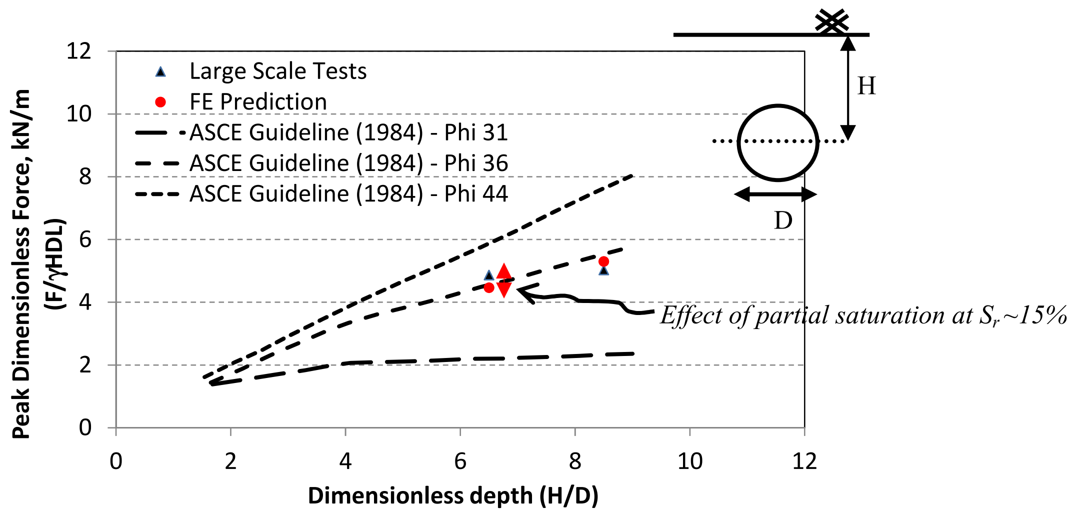


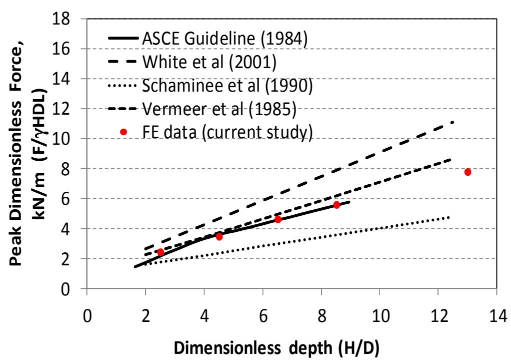
Fig. 7. Comparison of large scale test data and FE model predictions with ASCE guidelines for peak dimensionless force vs dimensionless depth

Response prediction for pipes buried in different sand initial densities

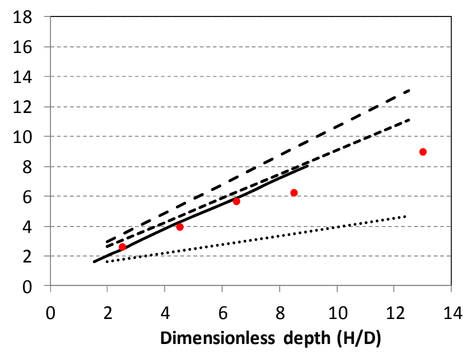
Further FE analyses were conducted on the basis of Chiba sands at different dry densities and pipe embedded depths to investigate the uplift resistance of soils. Dry densities were selected at effective friction angles of 36^{o} and 44^{0} from triaxial test data obtained from Robert, 2010. The triaxial test data showed that the corresponding dry densities are 1.43g/cm^{3} and 1.55g/cm^{3} at the relative dilatancy indices (Bolton, 1986) of 0.6 and 3 (at mean effective stress of 10kPa) respectively.

The results of FE analyses are compared with previous analytical solutions as well as to ASCE (1984) guideline predictions in Fig.8 a & b for different dry densities. It can be noted that the peak dimensionless load obtained from FE analyses agree well with ASCE predictions in general. However, the peak soil resistance for denser soils (i.e. ϕ'=44^{o}) shows a decrease in the increase of uplift resistance for deeply embedded pipe. This is because the different soil failure mechanisms at deep embedded pipes (deep-seated failure) in contrast to general shear failure of soil at shallow embedded pipes.

The predictions from the analytical models show a substantial difference when compared to ASCE guideline prediction and current FE analysis results. None of the analytical solutions are capable in capturing the non-linear increase of uplift soil resistance induced by different failure mechanisms at shallow and deeply embedded pipes in Chiba sand. The solutions proposed by White et al. (2001) and Schaminee et al. (1990) over-predict and under-predict the peak dimensionless force respectively, compared to ASCE and FE analysis results. The solution by Vermeer and Sutjiadi (1985) predicts similar peak dimensionless loads to ASCE and FE predictions for pipes buried at shallow depths, however it is unable to capture the different uplift resistances induced by varying soil failure mechanisms at deep embedded pipes.



(a) $\phi' = 36^o$



(b) $\phi' = 44^o$

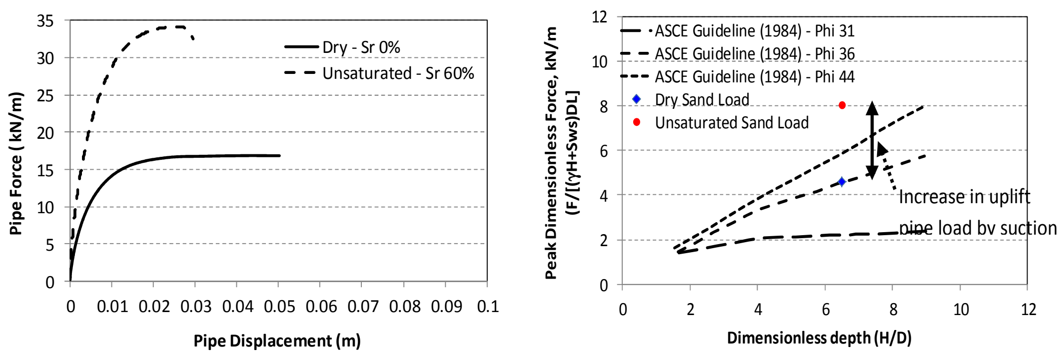
Fig. 8. Peak dimensionless force vs dimensionless depth

Response prediction for pipes buried in unsaturated soils

Previous studies have showed that the pipe behavior in lateral soil displacement under unsaturated condition can be substantially different when compared with the behavior under dry soil conditions (Robert, 2010; Robert and Soga, 2013; Robert et al, 2015). These studies showed that the lateral pipe loading under unsaturated condition can be substantially higher (more than a factor of 2) when compared to the loading in dry sand when tested at similar dry densities.

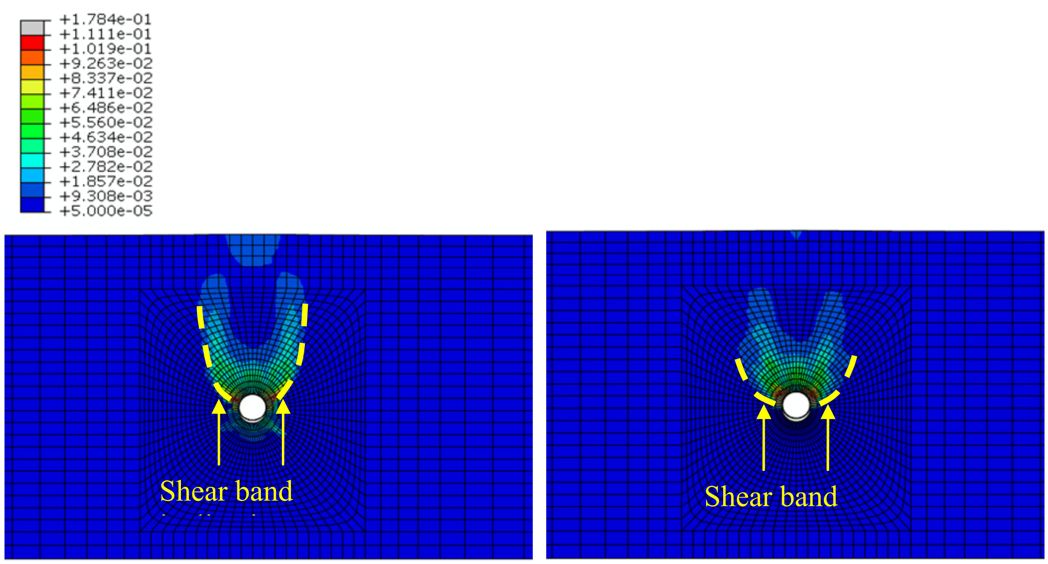
Analyses were conducted to investigate the behavior of pipes under uplift displacement in unsaturated soils. The initial dry density assumed for the soils is based on $\phi' = 36^o$ at a water saturation of 60%. Analyses were performed at H/D=6. Unsaturated Nor-Sand model was developed and calibrated for unsaturated Chiba sand (Robert, 2010; Robert and Soga, 2013; Robert et al, 2015). The results of the analyses are showed in Fig. 9-10.

The uplift resistance of the pipeline under unsaturated sand condition (i.e S_r=60%) has resulted significantly higher load when compared to the pipe loading obtained under dry sand condition as showed in Fig. 9. The suction induced apparent cohesion effect of Chiba sand has dramatically increased the uplift resistance of the pipeline when compared to dry condition (Robert, 2010). This can be further elaborated using the comparison of the deformation mechanisms between dry and unsaturated sand models. The deformation mechanism observed for unsaturated sand model showed broader shear band inclination compared to the soil deformation observed for dry sand model (Fig.11). This resulted for the peak dimensionless load to increase by a factor of ~2 for uplift loading in unsaturated sand (at 60% water saturation) when compared to the loading in dry sand at the same dry density of soil (Fig. 9b).



(a) Uplift resistance Vs displacement **(b) Peak Dimensionless Force Vs Dimensionless depth**

Fig. 9. Loading response of pipes buried in dry and unsaturated (60% water saturation) soils – H/D=6.0



(a) Dry Model (b) Unsaturated Model

Fig. 10. Deformation mechanisms (shear strains) of dry and unsaturated (S_r=60%) FE model (H/D=6)

CONCLUSION

The uplift resistance from the soil cover is a vital design parameter for HTHP pipelines as well as for the pipelines with high risk of exposure to earthquakes. This is due to the significant cost associated with the burial depth requirement in addition to the socio-economic losses that could result in the event of pipeline failures. Hence, for effective asset design and management, it is important to determine the accurate estimation of the uplift resistance from the soil cover above the pipeline. This paper presents full-scale uplift results and finite-element parametric studies conducted to investigate the effects of dimensionless cover heights (soil cover height to diameter ratio), soil relative density and moisture content on the peak uplift resistance of pipes subjected to vertical displacement in coarse-grained soils. Full scale experimental results were compared with the FE results for validation. The results from the current study are compared to ASCE design guidelines as well as available analytical models which predict the uplift resistance of soil.

Results from the current study revealed that the uplift resistance for pipes buried in dry soils agrees well with ASCE predictions in general. However, the uplift resistance of deeply embedded pipes (H/D>6) in denser soils are lower than the ACSE prediction. This is due to the different soil failure mechanism that occurs during the uplift of deeply embedded pipelines (deep-seated failure) in contrast to general shear failure of soil at shallow embedded pipes. None of the analytical solutions are capable in capturing the non-linear path of uplift soil resistance vs pipeline displacement or the transition of failure mechanisms at shallow and deeply embedded pipes.

The uplift resistance of the pipeline under partially-saturated sand condition (S_r=60%) is significantly higher when compared to the identical dry sand condition. The suction induced apparent cohesion effect of Chiba sand dramatically increases the uplift resistance of the pipeline when compared to dry condition. The peak dimensionless uplift load was increased by a factor of ~2 for uplift loading in partially saturated sand when compared to the uplift resistance in dry sand and ASCE guideline prediction at the same dry density of soil. The current study is being continued to investigate and quantify the partial saturation effects, burial depths and pipeline diameters on uplift resistance of soils.

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