A generalised framework to predict undrained uplift capacity of buried offshore pipelines

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<th>Journal:</th>
<th>Canadian Geotechnical Journal</th>
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<tr>
<td>Manuscript ID</td>
<td>cgj-2016-0153.R2</td>
</tr>
<tr>
<td>Manuscript Type:</td>
<td>Article</td>
</tr>
<tr>
<td>Date Submitted by the Author:</td>
<td>28-Jul-2016</td>
</tr>
<tr>
<td>Complete List of Authors:</td>
<td>Maitra, Shubhrajit; Indian Institute of Technology Bombay, Civil Engineering Chatterjee, Santiram; Indian Institute of Technology Bombay, Civil Engineering Choudhury, Deepankar; Indian Institute of Technology Bombay,</td>
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<tr>
<td>Keyword:</td>
<td>Pipelines, Numerical modelling, Offshore engineering, Uplift, Undrained capacity</td>
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Revised manuscript submitted to Canadian Geotechnical Journal on 28/07/2016

Manuscript ID: CGJ-2016-0153

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No. of words: 5708 (without abstract and references)
No. of tables: 4
No. of figures: 16
A generalised framework to predict undrained uplift capacity of buried offshore pipelines

A generalised framework to predict undrained uplift capacity of buried offshore pipelines

Shubhrajit Maitra, Santiram Chatterjee and Deepankar Choudhury

Abstract

Estimation of undrained uplift capacity is essential for the determination of optimal burial depth of buried offshore pipelines. However, a generalised prediction model that incorporates various factors influencing this capacity is scarce in the literature. In this paper, results from a series of small strain finite element analyses are presented that explore the effects of pipe embedment, pipe-soil interface roughness, interface tensile capacity, soil shear strength and unit weight on pipe uplift response. From the study, a simple method to predict the undrained upheaval resistance of buried pipelines for any practical range of pipeline and soil parameters is proposed. Factors associated with transition in failure mechanism with embedment are also examined. The numerical model is validated by comparing the results with available analytical and experimental data. Large deformation finite element analyses were also performed independently for a few cases in order to justify the applicability of small strain methods in modelling pipe upheaval. Accuracy of the model for generalised shear strength profile is then examined by considering practical values of parameters over broad ranges. The proposed methodology gives results with maximum error less than 8% for all ranges of parameters and hence can be adopted in design practices.

Keywords: Pipelines; numerical modelling; offshore engineering; uplift; undrained capacity.

Words: 5708, Tables: 4, Figures: 16.
Introduction

Oil and gas industries, particularly those located offshore, have shown significant growth over the past few decades. Offshore pipelines are often termed as ‘arteries’ of these industries as they serve as means of conveyance of hydrocarbons within a field as well as to shore. The pipelines are either laid on the seabed or buried inside trenches which are either pre-cut or formed during laying processes. Burying the pipelines into the seabed diminishes hydrodynamic wave forces acting on them. Damages incurred due to fishing and trawling activities can also be prevented by burying the pipelines. Apart from these, soil cover due to burial provides thermal insulation to the pipelines. This insulation is essential as the extracted hydrocarbons need to be maintained at high temperature and pressure to prevent hardening of the wax portion ensuing ease of flow. High operating temperature and pressure induce axial and hoop stresses within the pipe cross-section, which are resisted by the bearing and frictional resistances of the surrounding soil. However, when the axial stresses increase significantly, the pipe may tend to buckle either in lateral or vertical direction. For buckling in upward direction (upheaval buckling), resistance can be attributed to the shearing strength and self-weight of the soil cover.

Estimation of optimal burial depth and upheaval resistance for buried pipes is very essential because overestimation of this depth may result in huge capital expenditures incurred from additional trenching, whereas underestimation may threaten integrity of the entire system leading to substantial losses associated with repairing. In the past, attempts have been made by several researchers to study soil-structure interactions for assessing ultimate soil resistances and failure mechanisms for circular buried objects using classical plasticity theory (Randolph and Houlsby 1984; Martin and Randolph 2006; Martin and White 2012). Numerical modelling has been widely used to estimate uplift capacities for buried pipelines.
A generalised framework to predict undrained uplift capacity of buried offshore pipelines

(Aubeny et al. 2005; Newson et al. 2006; Thusyanthan et al. 2011). Various experimental works using physical model tests either at normal gravity (Schaminee et al. 1990; Palmer et al. 2003; El-Gharbawy 2006; Liu et al. 2013; Liu et al. 2015; Chen et al. 2016) or under enhanced gravity conditions in geotechnical centrifuge (Bransby et al. 2002; Palmer et al. 2003; Cheuk et al. 2007; Thusyanthan et al. 2008) have also been performed to study the response of soil during pipe uplift. Empirical relations proposed from most of these works are case specific and valid strictly within the domain in which they were carried out. During upheaval, the soil response may be drained or undrained depending on permeability of the soil and the rate of loading. Martin and White (2012) highlighted the shortcomings of DNV RP-F110 (DNV 2007), which is in vogue in current industrial practice for predicting undrained uplift capacity relevant for fine grained soils. Previous researchers have pointed out that uplift failure mechanism undergo a transition from ‘global failure mode’ to ‘local failure mode’ with increase in pipe embedment (Newson et al. 2006; El-Gharbawy 2006; Martin and White 2012). In global failure mode, the mechanism reaches the seabed surface whereas, in local failure mode, a localised circumferential flow of soil occurs around the pipe during uplift. However, a proper guideline on the type of mechanism a pipe is likely to undergo during failure is missing in literature. A generalised framework or prediction model for estimating undrained uplift capacity of buried pipelines for various shear strength profiles, pipe embedment, unit weight of soil, soil heterogeneity factor, pipe-soil interface roughness and tensile capacity also does not exist in literature.

In this paper, a comprehensive study has been carried out to estimate undrained uplift capacity of buried pipelines using finite element software ABAQUS 6.13 (Dassault Systèmes 2013). Analyses were performed for several values of soil cover height ratio \( (H/D) \), normalised submerged unit weight of soil \( (\gamma'D/\gamma_u) \) for soils with uniform undrained shear strength \( (\gamma_u) \) and \( \gamma'/k \) for soils with \( s_u \) proportional to depth, non-dimensional soil
heterogeneity factor \( (\kappa = kD/s_{um}) \) and extreme values of pipe-soil interface roughness coefficient \( (\alpha) \) and interface tensile capacity \( (T) \). Fig. 1 shows a schematic representation of the problem geometry and shear strength profiles considered in the present study. A simple but generalised framework that incorporates the effects of all these parameters is proposed for prediction of undrained upheaval resistance of soil. All the results are presented in non-dimensional form so that they can be implemented in design practice.

**Finite Element Methodology**

**Material model, mesh optimisation and boundary conditions**

Small strain finite element (SSFE) analyses were carried out by creating a two dimensional plane strain model in ABAQUS. Mohr-Coulomb elastic perfectly plastic constitutive relationship with zero friction angle (essentially the Tresca failure criteria) was adopted to model the undrained shear strength of clayey soils. Mesh discretisation was done using six-noded quadratic plane strain triangular elements (CPE6) available in ABAQUS standard library. Fig. 2 shows details of a typical mesh along with boundary conditions and model dimensions. The bottom boundary is restrained both horizontally and vertically, whereas the side boundaries are restricted against any lateral movement. Optimum extent of the soil domain in all directions was considered by ensuring minimal boundary effects on failure mechanisms. In order to obtain load factors with negligible errors within reasonable computation time, mesh optimisation in terms of mesh density at both near and far field locations were carried out. Minimum element size adopted was 0.01% of pipe diameter \( (D) \) near pipe-soil interface.

**Details of analyses**
A detailed parametric study by varying the influencing parameters in a systematic manner was carried out to understand the undrained uplift behaviour of buried pipelines. Total stress analyses assuming Poisson’s ratio (ν) of 0.499 (~0.50), replicating essentially the constant volume condition, were performed for soils with $s_u$ constant ($s_u = s_{um}$, $k=0$) and proportional to depth ($s_{um}=0$, $s_u=kz$). However, in order to overcome numerical limitations, a very small value of $s_{um}$ (=0.001kPa) was assigned for soils with $s_u=kz$. Modulus of elasticity of soil ($E$) at a particular embedment was considered as 500 times of $s_u$ at that location (i.e., stiffness index, $E/s_u=500$). Undrained uplift capacity is independent of $E/s_u$ for elastic perfectly plastic constitutive relationship of soil. However, the pipe displacement at which this capacity gets mobilised is a function of $E/s_u$. In order to study the effect of submerged unit weight of soil ($\gamma'$) on failure mechanisms, several values of $\gamma'$ (uniform along depth) were considered. Pipe was assumed to be rigid and weightless. Offshore pipelines are generally made of steel and are at times coated externally with thermally insulating and anti-corrosive agents. Depending on this and also on soil particle size, the pipe-soil interface can vary from ‘perfectly smooth’ to ‘fully rough’. Roughness of pipe-soil interface is characterised by friction coefficient $\alpha$ which is the ratio of mobilised shear strength at the interface to the shear strength of surrounding soil. Interface was assumed to be either smooth ($\alpha=0$) or fully rough ($\alpha=1$). During uplift process, a gap may get formed below pipe if adequate interface tensile strength ($T$) is not mobilised. Extreme values of $T$ were considered in the present study ($T=0$ corresponds to no tension (NT) while $T=\infty$ corresponds to full tension (FT) mobilised at interface). Thus, the following four cases were considered: (i) Smooth FT: Smooth pipe with unlimited tensile capacity at interface, (ii) Smooth NT: Smooth pipe with zero interface tension, (iii) Rough FT: Rough pipe with infinite interface tensile strength, and (iv) Rough NT: Rough pipe with no tension developed at interface. To model Rough FT conditions, the pipe surface was tied to the soil interaction surface using ‘tie’ constraint in ABAQUS,
whereas Rough NT conditions were replicated using ‘penalty’ method of friction formulation by limiting shear stress at interface to $s_u$ of the environing soil mass. Table 1 shows a summary of the various input parameters in non-dimensional form. Very high values of $H/D$ have been considered, even outside the practical range commonly implemented in field, in order to establish the extreme limits of the prediction model. Pipes were subjected to upward displacements, typically in the range of 0.10-0.50 times of $D$, until fully plastic conditions were developed.

**Validation of the numerical methodology**

*Comparison of SSFE results with earlier studies*

Fig. 3(a) shows a comparison of obtained undrained uplift capacity factor ($N_u$) for a few cases with results available in literature. Randolph and Houlsby (1984) provided analytical solutions using classical plasticity theory for local shear failure mechanism during lateral movement of pile at deep embedment in weightless soil. The limiting values of load factors proposed were $6+\pi$ for perfectly smooth interface and $4\sqrt{2}+2\pi$ for fully rough interface which are in well agreement (see Fig. 3(a)) with those obtained in the present study (9.2 and 12 for smooth and rough pipes respectively).

Martin and White (2012) had carried out a similar study using finite element limit analysis software OxLim for obtaining $N_u$ for $H/D \leq 4$. The results obtained from the present study are in well agreement with their work for all the cases except for Rough NT cases (see Fig. 3(a)). The obtained load factors showed maximum deviation of 0.5% for Smooth NT, Smooth FT and Rough FT cases. However, for the cases where the pipe embedment is less than the embedment of limiting capacity, $N_u$ obtained from the present analyses for Rough NT conditions are considerably lower than Martin and White (2012) because OxLim provides...
solutions based on classical plasticity theory assuming a different interface behaviour that arises from the requirement of normality condition. Normality implies that yield surface is the plastic potential and normals drawn to it give the directions of flow. The consequence of this assumption is that even when separation occurs (i.e., normal stress developed at pipe surface is zero), full shear strength can be mobilised (as described in Houlsby and Puzrin 1999; Chatterjee 2012). However, such assumption is not practical as in the case of a gap getting formed, neither shear stress is generated at pipe-soil interface nor can it be transmitted. Under such scenarios, plasticity theory solutions overestimate load factors.

Limited experimental studies have been carried out in the past using geotechnical centrifuge to study undrained uplift capacity of pipelines. Thusyanthan et al. (2008) conducted centrifuge tests at 30g and obtained undrained uplift factors of 10.70 and 10.43 at $H/D=4.02$ and 4.98 respectively for pipe surface with intermediate roughness and $\gamma'D/s_u=0.95$. For comparison with these results, some additional numerical analyses were carried out using the equivalent prototype values of the problem in the centrifuge. $N_u$ obtained from the centrifuge tests is found to lie between the uplift capacity factors obtained numerically for smooth and rough pipes (see Fig. 3(a)).

**Comparison of small strain and large deformation FE methodology**

In the small strain analyses, undrained uplift capacities at different initial embedment levels were obtained by placing the pipe at that depth. Though SSFE analyses were carried out for this study, pipeline typically undergoes large displacement during upheaval buckling. Few cases of large deformation finite element analyses (LDFE) adopting “remeshing and interpolation technique with small strain” (RITSS) (Hu and Randolph 1998a, 1998b) were also performed to track the continuous uplift resistance during the upward movement of pipe from the initial embedment location. Detailed information on the RITSS approach adopted
A generalised framework to predict undrained uplift capacity of buried offshore pipelines


here can be found in Wang et al. (2010) and Chatterjee et al. (2013). Fig. 3(b) and 3(c) show uplift resistance response from LDFE analyses with initial $H/D$ of 1, 3, and 5 for smooth and rough pipes respectively for $T=0$ in weightless soil. Uplift factors obtained from SSFE analyses are also shown in these figures for comparison. LDFE curves show that peak uplift resistances are mobilised within small pipe displacement and in all the cases, peak resistances from LDFE results merge nicely with the SSFE resistances. Thus, mesh distortion had negligible effect on obtained SSFE results for undrained uplift capacities. After reaching peak, the uplift resistance drops due to loss of embedment. However, the post peak behaviour from LDFE and SSFE tend to deviate because SSFE analyses do not capture the formation of heave during upward movement. The objective of the present study is to predict the maximum resistance a pipe will experience at a particular depth during uplift. Hence, SSFE analyses were preferred as it is computationally much less expensive than LDFE.

Results and discussions

Soils with uniform undrained shear strength along depth ($s_u = s_{um}$, $k = 0$)

Undrained uplift capacity ($V_u$) is dependent on both soil effective unit weight and shear strength. As $V_u$ is proportional to $s_u$, non-dimensional undrained uplift capacity factors ($N_u$) were obtained for all the cases by normalising $V_u$ with respect to $s_u D$ ($N_u = V_u / s_u D$). Fig. 4 illustrates the obtained $N_u$ for smooth and rough pipes under NT (dashed lines) and FT (solid lines) conditions for values of $\gamma'D/s_u$ ranging from 0 to 5. Fig. 5 shows the differences in failure mechanisms for rough pipes with the change of embedment, soil unit weight and interface tensile capacity. The displacements at various regions of soil domain ($u_{soil}$) have been normalised by the displacement applied to the pipe ($u_{pipe}$). From these figures, it becomes evident that the uplift failure mechanism undergoes a transition from global failure mode to local failure mode with increase in soil cover height ratio ($H/D$). The main
A generalised framework to predict undrained uplift capacity of buried offshore pipelines


distinguishing feature between these two types of mechanism is that in global shear failure (see Fig. 5(a) and 5(c) – 5(e)), the failure mechanism reaches the seabed surface, whereas in local shear failure (see Fig. 5(b) and 5(f)), the mechanism is spread over a smaller area with localised circumferential flow of soil during uplift. It can also be observed that under FT conditions, no breakaway (NBA) i.e., no separation can occur between bottom of the pipe and soil beneath (see Fig. 5(a) – 5(b)) because of unlimited interface tensile strength. However, for NT conditions, breakaway (BA) occurs between pipe and soil at ‘shallow’ embedment (see Fig. 5(c) – 5(e)) and the soil fails globally. At ‘deep’ embedment, separation cannot occur even under NT conditions (see Fig. 5(f)) and the observed failure mechanism is identical to that under FT conditions (compare Fig. 5(f) with 5(b)) suggesting that a transition in failure mechanism occurs with increase in embedment. It is to be noted that in Fig. 4 the dashed lines and the corresponding solid lines merge at embedment for which this transition occurs and simultaneously $N_u$ reaches its limiting value because of similar mechanisms for NT and FT cases beyond this depth. The embedment ratio of pipe invert ($w/D$) at which this transition takes place for NT conditions has been termed as limiting depth ratio ($w_{\text{limit}}/D$) which depends on $\gamma'D/s_u$ and $\alpha$. This depth has been expressed in terms of depth of pipe invert, because breakaway occurs at rear of the pipe.

For FT conditions, $H/D$ at which $N_u$ reaches its maxima depends on $\alpha$ only and is independent of $\gamma'D/s_u$ (see Fig. 4). The soil cover height ratios at which $N_u$ reaches its maxima are approximately 1.2 and 2.2 for smooth and rough pipes respectively. It can also be noted that the failure mechanism undergoes transition from global to local mode at this embedment (compare Fig. 5(a) and 5(b)) and correspondingly the curve for $N_u$ versus $H/D$ reaches a limiting value as the volume of soil in sheared zone becomes constant and independent of embedment. These load factors are in well agreement with those proposed by Randolph and Houlsby (1984) for weightless soil as discussed earlier. However, with increase in $\gamma'$, $V_u$
A generalised framework to predict undrained uplift capacity of buried offshore pipelines


decreases by $\gamma' A_s$ (see Fig. 4), where $A_s$ is the submerged cross-sectional area of the pipe. This occurs due to the buoyancy effect of soil which assists pipe uplift because of predominant soil movement in the direction of gravity.

For NT conditions, separation occurs between bottom of pipe and soil for $w < w_{\text{limit}}$. Soil movement occurs predominantly in the upward direction. Thus, work is done against gravity resulting in increase in $N_u$ with increase in $\gamma'D/s_u$. However, for higher values of $\gamma'$, the overburden stress may get so high that it resists formation of gap below the pipe. At $w \geq w_{\text{limit}}$, separation cannot occur and failure mechanism is identical to that under FT conditions. Thus, the curves for NT and FT cases merge beyond $w_{\text{limit}}$. Higher the value of $\gamma'D/s_u$, lower is the value of $w_{\text{limit}}$. Hence, estimation of $w_{\text{limit}}$ serves to be crucial in prediction of $N_u$.

Soils with undrained shear strength proportional to depth ($s_u = kz$, $s_{um} = 0$)

In-situ $s_u$ of normally consolidated soils often increases linearly with depth starting from a near-zero value at the mudline. For this case, estimation of $N_u$ becomes further complex due to the heterogeneity of soil shear strength. Determination of $N_u$ requires $V_u$ to be normalised by $s_uD$. However, for this type of shear strength profile, $s_u$ varies with depth. The practice that is commonly adopted is to normalise $V_u$ with respect to $s_{u(centre)}D$ or $s_{u(invert)}D$. Such simplified normalisation is not proper and leads to unrealistic values of uplift capacity factors for most of the cases as discussed next.

The embedment at which $N_u$ reaches its limiting value ($w_{\text{limit}}$) depends on $\gamma'D/s_{u(centre)}$ i.e., on $(\gamma'/k)(D/(w-0.5D))$ and $\alpha$. Hence, analyses were performed by varying the non-dimensional term $\gamma'/k$ over wide ranges for both smooth and rough pipes. Fig. 6 shows the obtained $V_u$ normalised with respect to $s_{u(centre)}D$ for smooth and rough pipes under FT conditions in soils with $s_u = kz$. Few examples of uplift mechanism for smooth pipes are depicted in Fig. 7. From
A generalised framework to predict undrained uplift capacity of buried offshore pipelines

Fig. 7(b), it can be seen that for FT conditions at ‘deep’ embedment, local shear failure mechanism occurs similar to soils with uniform $s_u$. For this case with weightless soil, $V_u/s_{u(centre)}D$ yields the same $N_u$ as for soil with uniform $s_u$ (9.2 and 12 for smooth and rough pipes respectively) (see Fig. 6) as the average mobilised $s_u$ within the sheared soil is very close to $s_{u(centre)}$. In case of weighty soils, the limiting values of $N_u$ are $(9.2−γ'A/s_{u(centre)}D)$ and $(12−γ'A/s_{u(centre)}D)$ for smooth and rough pipes respectively. However, as $s_u$ increases with depth, the limiting value of $N_u$ also increases with $H/D$ due to decrease in $γ'A/s_{u(centre)}D$. For the cases where limiting resistance is not mobilised (e.g. $γ'/k=0$, $H/D≤2$ in Fig. 6), the plot of $V_u/s_{u(centre)}D$ versus $H/D$ does not show an ordered pattern as was observed for soils with uniform $s_u$. This is because at shallow embedment the failure mechanism spreads more towards the bottom of the pipe (similar to reverse end bearing phenomenon), resulting in average $s_u$ mobilised within the sheared soil being greater than $s_{u(centre)}$. Thus, in some cases, $V_u/s_{u(centre)}D$ results in unrealistic values of uplift factors being more than the limiting values (e.g. $γ'/k=0$, $H/D=0$). This necessitates the determination of average mobilised $s_u$ for establishing a prediction model. It can also be observed from Fig. 6 that obtained uplift capacities have negative values for some of the cases (e.g. $γ'/k=10$, $H/D=0$) where the soil is unable to stand on its own weight because of high $γ'$ and very low $s_u$.

Fig. 8 and Fig. 9 present the undrained uplift capacity factors (normalised with respect to $s_{u(centre)}D$) under NT conditions for smooth and rough pipes respectively. From the figures, it is observed that $V_u/s_{u(centre)}D$ increases with depth for all the cases as expected. However, a concerning observation that can be made is that with increase in $γ'/k$, $V_u/s_{u(centre)}D$ shows an increase followed by decrease for all depths. For lower $γ'/k$, separation between pipe bottom and soil occurs and weight of the soil above the pipe resists uplift. In the case of higher $γ'/k$, $γ'D/s_{u(centre)}$ is significantly large at shallower depth. Thus, even though no interface tension is mobilised, breakaway cannot occur even at smaller $H/D$ (see Fig. 7(e)). Hence because of
buoyancy effect, $N_u$ decrease due to substantial increase in $\gamma' A_s/s_{u(centre)} D$ at shallow embedment. However, with increase in embedment, $\gamma' A_s/s_{u(centre)} D$ decreases which results in an increase in $N_u$. Normalisation of uplift capacity using $s_{u(centre)} D$ is not justifiable for all the cases where breakaway occurs. Under such circumstances soil lying above the pipe is displaced and the average $s_u$ of the displaced soil is lower than $s_{u(centre)}$. All these factors highlight the complexity of the problem for soils with $s_u=kz$.

**Formulation of the undrained uplift capacity prediction model**

The previous studies and current design practices have several limitations and do not provide a proper guideline/generalised framework for estimation of undrained uplift capacity for buried pipelines. However, as described in the preceding sections, pipe-soil interaction during uplift is a complex phenomenon involving many parameters. In this section, a prediction model is proposed considering the effects of all the relevant parameters.

**Full Tension (unlimited interface tensile capacity)**

Analyses have been performed for $H/D\geq0$ as the study is concerned with buried pipes only and on-bottom pipelines are not taken into account. However, in order to formulate a prediction model under Full Tension (FT), it becomes essential to obtain $N_u$ for pipe placed on the seabed (i.e., $w/D=0$) for weightless soil. Uplift failure mechanism under FT conditions for pipe placed at very shallow embedment is identical to reverse end bearing mechanism. Additional analyses were performed for small values of $w/D$ in weightless soil with uniform $s_u$ to obtain $N_{u(weightless)}$ at $w/D=0$. The pipe was assumed to be wished-in-place without the formation of any surface heave on the seabed. Table 2 shows the obtained results from these analyses for both smooth and rough pipes. It can be noted that the uplift capacities at various embedment have been normalised by $s_u D'$ to obtain $N_{u(weightless)}$. Here, $D'$ is the effective...
A generalised framework to predict undrained uplift capacity of buried offshore pipelines

diameter which is the maximum chord length in the embedded portion of the circular pipe cross-section. For pipes embedded more than 50% of its diameter \( D' = D \), whereas for \( w/D \leq 0.5 \), \( D' = 2\sqrt{(w(D-w))} \). \( N_u \) for pipe placed at the surface of weightless soil (\( N_{u0} \)) was obtained approximately as 4.5 and 5.0 for smooth and rough pipes respectively (see Table 2).

As discussed in the preceding sections, for weighty soil, the reduction in \( V_u \) due to buoyancy effect is exactly equal to \( \gamma' A_s \). Thus, by negating the effect of buoyancy under FT conditions, all the curves for various values of \( \gamma'D/s_u \) in Fig. 4 and \( \gamma'/k \) in Fig. 6 reduce to their corresponding curves for weightless soil as shown in Fig. 10. \( V_u \) for weightless soil or \( V_u \) after negating the buoyancy effect for weighty soil is termed as geotechnical resistance. \( V_u \) for all the cases can be normalised by \( s_{u,eff}D \) to obtain \( N_u \), where \( s_{u,eff} \) is the average \( s_u \) mobilised within the sheared soil. So, the uplift capacity factor is given by

\[
N_u = \frac{V_u}{s_{u,eff}D} = N_{u(weightless)} - \left( \frac{\gamma'}{s_{u,eff}D} \right) A_s
\]

Expression for \( N_{u(weightless)} \) is obtained by means of curve fitting using error minimisation through method of least squares. Thus, \( N_u \) under FT conditions is finally expressed as

\[
N_u = \frac{V_u}{s_{u,eff}D} = N_{u0} + \left( N_{u(limit)} - N_{u0} \right) \left[ 1 - \exp \left\{ -0.4 \left( 1 + H/D \right)^{\beta_1} \right\} \right] - \left( \frac{\gamma'}{s_{u,eff}D} \right) A_s
\]

Here, \( N_{u(limit)} \) are the limiting values of \( N_{u(weightless)} \) as proposed by Randolph and Houlsby (1984) and \( \beta_1 \) is a curve fitting parameter (see Table 3). For soils with uniform \( s_u \), \( s_{u,eff} = s_u \), whereas for soils with \( s_u = k \), \( s_{u,eff} \) is defined by

\[
s_{u,eff} = f_{NB4} k \left( \frac{H + D}{2} \right)
\]
Here, \( f_{NBA} \) is a factor for obtaining \( s_{u,eff} \) for the ‘\( k_z \)’ component of \( s_u \). Since \( s_{um} \approx 0 \) for these cases, the factor \( f_{NBA} \) has been back-calculated at each embedment by taking ratio between \( V_u/s_u(centre)D \) for weightless soils with \( s_u=k_z \) and uniform \( s_u \). Finally, \( f_{NBA} \) is expressed using eq. (4) with \( \lambda_1 \) and \( \lambda_2 \) being the curve fitting parameters (see Table 3) and is plotted as a function of \( H/D \) in Fig. 11(a).

\[
(4) \quad f_{NBA} = \frac{1}{1 - \exp\left(-\lambda_1 \left(1 + \frac{H}{D}\right)^{\lambda_2}\right)}
\]

It was observed that at ‘shallow’ embedment, majority of the displaced soil lied below centre of the pipe as the mechanism was similar to reverse bearing capacity failure. Thus, for soils with \( s_u=k_z \), \( s_{u,eff} \) is greater than \( s_u(centre) \) at smaller \( H/D \) resulting in \( f_{NBA} \geq 1 \) (see Fig. 11(a)). However, for local shear failure at ‘deep’ embedment, \( s_{u,eff} \) becomes equal to \( s_u(centre) \) as the failure mechanism is approximately symmetrical about horizontal axis passing through pipe centre resulting in \( f_{NBA} \) being equal to 1 for such cases. The transition in failure mechanisms with increase in embedment is nicely represented by this variation of \( f_{NBA} \).

**Estimation of limiting depth (\( w_{limit} \))**

Before prediction of \( N_u \) under NT conditions, it becomes essential to know whether breakaway occurs, as the failure mechanisms are entirely different depending on this. As pointed out in previous sections, \( (w_{limit}/D) \) is a function of \( \alpha \) and \( \gamma'D/s_u(centre) \). From numerous analyses performed for various \( \gamma'D/s_u(centre) \), \( (w_{limit}/D) \) are determined corresponding to the \( w/D \) at which \( N_u \) under NT and FT conditions become equal. In other words, \( w_{limit}/D \) is the minimum \( w/D \) at which separation cannot occur even under NT conditions. The obtained results are plotted (see Fig. 12) and for estimating \( w_{limit}/D \), a power model is proposed.
A generalised framework to predict undrained uplift capacity of buried offshore pipelines

\[
\left( \frac{W_{\text{limit}}}{D} \right) = \frac{\varphi}{s_1} \times \left( \frac{\gamma' D}{s_{u,\text{centre}}} \right)^{-\xi_2}
\]

The values of curve fitting parameters $\xi_1$ and $\xi_2$ presented in Table 3.

**No Tension (zero interface tensile capacity)**

For determination of $N_u$ under NT conditions, $w_{\text{limit}}$ needs to be first determined using eq. (5).

For $w \geq w_{\text{limit}}$, separation cannot occur and the failure mechanism under NT condition is identical to that under FT case. The corresponding $N_u$ can be computed using eq. (2) – (4).

However, for $w < w_{\text{limit}}$, breakaway occurs between pipe and soil. During uplift, shearing resistance developed in soil restricts the upward movement of pipe in conjunction with weight of the soil above the pipe. The component of uplift resistance ($\Delta V_u$) due to weight of soil can be computed using principal of virtual work. Consider a small segment of pipe-soil interface (shown by bold line in Fig. 13). The work ($\delta W$) done by the elemental pipe segment in displacing the soil above it by $\delta u$ is

\[
\delta W = \gamma' \left( H + \frac{D}{2} - \frac{D}{2} \sin \theta \right) \times \left( \frac{D}{2} \sin \theta d\theta \right) \times \delta u
\]

Thus, work done during uplift can be equated with $\Delta V_u$ as follows

\[
\Delta V_u \times \delta u = 2 \int_0^{\xi_2} \gamma' \left( H + \frac{D}{2} - \frac{D}{2} \sin \theta \right) \times \left( \frac{D}{2} \sin \theta \right) \delta u \, d\theta
\]

Equation 7 can be simplified to obtain $\Delta V_u$ and is expressed as

\[
\Delta V_u = \gamma' \left[ HD + D^2 \left( \frac{1}{2} - \frac{\pi}{8} \right) \right] = W_{\text{soil}}
\]
Hence, it can be observed from eq. (8) that $\Delta V_u$ is simply the weight of the vertical column of soil ($W_{soil}$) above the pipe (shown by shaded region in Fig. 13). Hence, when breakaway occurs under NT conditions, the curves for various $\gamma'D/s_u$ in Fig. 4 and $\gamma'k$ in Fig. 8 and Fig. 9 can be reduced to their corresponding curves for weightless soil by negating the buoyancy contribution and has been shown in Fig. 10. The undrained uplift capacity factors in weighty and weightless soils can be related as follows

$$N_{u(heavy)} = N_{u(weightless)} + \frac{W_{soil}}{s_{u,eff}D}$$

By error minimization using method of least squares, a prediction equation for $N_{u(weightless)}$ has been obtained and $N_u$ for NT cases where separation occurs is finally expressed as

$$N_u = \frac{V_u}{s_{u,eff}D} = N_{u(toal)} \left(1 - \frac{1}{1 + 0.2 \left(\frac{H}{D} + 0.5\right)}\right) + \frac{W_{soil}}{s_{u,eff}D}$$

Here, $\beta_2$ is a curve fitting parameter (see Table 3) and $W_{soil}$ can be computed using eq. (8).

For soils with constant $s_u$, $s_{u,eff}$=$s_u$, whereas for soils with $s_u$=$kz$, $s_{u,eff}$ can be obtained by

$$s_{u,eff} = f_{BA}k\left(H + \frac{D}{2}\right)$$

Here, $f_{BA}$ is a factor incorporating the effect of soil heterogeneity on $s_{u,eff}$ and is calculated for each embedment similar to $f_{NB}$ and is expressed as

$$f_{BA} = 1 - \exp\left(-\lambda_3 \left(1 + \frac{H}{D}\right)^{\lambda_4}\right)$$
Here, curve fitting parameters $\lambda_3$ and $\lambda_4$ are listed in Table 3. When breakaway occurs, soil above the level of pipe centre gets displaced. Thus, for $s_u=kz$, $s_u(\text{centre}) \geq s_u,\text{eff}$ and $f_{BA} \leq 1$. At infinite embedment, $f_{BA}=1$ as $w_{\text{limit}}$ for weightless soil is also infinity.

The obtained undrained uplift capacities ($V_{u(\text{obtained})}$) using ABAQUS are compared with those predicted by the proposed model ($V_{u(\text{predicted})}$) for soils with uniform $s_u$ and $s_u=kz$ and are represented in Fig. 14(a) and 14(b). It is observed that the error is within 10% for most cases. For few cases with very shallow embedment and lower soil unit weight, the errors exceeded 10% as the capacity is very low and small error involved in curve fitting magnifies the % error. Nevertheless, for all sets of practically encountered values of parameters, margin of error is quite low which is dealt with in the next section.

Soils with generalised undrained shear strength profile ($s_u=s_{um}+kz$, $s_{um} \geq 0$, $k \geq 0$)

In the preceding sections, analyses were carried out for the cases of uniform $s_u$ and $s_u=kz$. However, lightly overconsolidated soils with non-zero mudline shear strength is very commonly encountered. Additional analyses were performed for a few values of normalised pipe embedment in soils with generalised shear strength profile, $s_u=s_{um}+kz$ to check whether the prediction model described above for uniform ($s_u=s_{um}$) and linearly increasing shear strength profile ($s_u=kz$) can be superimposed to predict the uplift capacity for these general cases. For these cases, the effective shear strength can be calculated as:

(i) For full tension and no tension without breakaway ($w \geq w_{\text{limit}}$),

\[
(13) \quad s_u,\text{eff} = s_{um} + f_{NBA}k\left[H + \frac{D}{2}\right]
\]

(ii) For no tension with breakaway ($w < w_{\text{limit}}$),

\[
(14) \quad s_u,\text{eff} = s_{um} + f_{BA}k\left[H + \frac{D}{2}\right]
\]
For these set of analyses, values of $H/D$, $s_{um}$ and $k$ were chosen over wide ranges which are often met with in reality. Values of $k$ and $s_{um}$ were taken considering large variation of soil heterogeneity factor ($\kappa$) often observed in offshore soil conditions. Table 4 shows the combinations of input parameters considered for generalised undrained shear strength profile of soil. Undrained uplift capacity were calculated for all combinations ($V_{u(predicted)}$) using the proposed model (eq. (2), (4), (5), (8), (10), (12) – (14)). Fig. 14(c) shows the comparison of $V_{u(predicted)}$ with the obtained values ($V_{u(obtained)}$) using ABAQUS. Results show that error for the combinations considered is <5% for most of the cases whereas, the maximum error is within 8%. Thus, the proposed undrained uplift capacity prediction model gives correct and critical results and hence can be adopted in design practice.

Comparison of proposed methodology with DNV (2007) guidelines

Fig. 15 shows a comparison of the proposed prediction methodology with DNV (2007) guidelines that are widely used for predicting $V_u$. Several shortcomings do exist in the guidelines as evident from the figure. These are summarised as follows:

(i) DNV (2007) suggests that undrained uplift capacities are governed by two main failure mode: “Local soil failure” and “Global soil failure”. As per DNV (2007), local soil failure mode corresponds to FT conditions, whereas soil fails globally when breakaway occurs under NT conditions. However, it does not state that even under NT conditions, breakaway may not occur (i.e., when $w > w_{limit}$) and a transition in failure mechanism takes place with increase in $H/D$.

(ii) DNV (2007) assumes a simple vertical slip model for global soil failure mode. Thus, when breakaway occurs, capacities become independent of $\alpha$. This assumption is not realistic as slip surfaces may not be vertical (as evident from Figs. 5 (c) – (e) and 7 (c) – (d)). The
length of the failure surface varies with $\alpha$ and thus the capacities are more for rough interfaces (upto 30%) as compared to smooth interfaces.

(iii) As also pointed out by Martin and White (2012), vertical slip model overestimates $V_u$ (see Fig. 15(a) and 15(b)) and is thus unconservative.

(iv) Under FT conditions, DNV (2007) does not consider the detrimental effect of soil buoyancy on $V_u$. Moreover, it recommends use of an empirical factor $\eta$ (which lies in the range 0.55-0.8 and is typically taken as 0.65) for obtaining $V_u$. The use of this factor underestimates $V_u$ (upto 35%) in soils with low $\gamma'D/s_{u(centre)}$.

(v) DNV (2007) suggests that limiting capacities are mobilized under FT conditions at $H/D = 4$. However, from present study it is evident that limiting capacities are attained at $H/D$ roughly equal to 1.2 and 2.2 for smooth and rough pipes respectively.

Conclusions

The present study deals with development of a simple unified prediction model for undrained uplift capacities of buried pipes in soil. Analyses were carried out for the following four cases: a) Smooth NT, b) Rough NT, c) Smooth FT and d) Rough FT using finite element based software ABAQUS. Additional LDFE analyses carried out for few cases indicated that the SSFE model was able to capture peak uplift resistances though the problem involves large displacements. The results were validated with presently available theoretical and experimental studies in the literature. From the present study, it has been found that for $w < w_{limit}$, plasticity theory solutions overestimate $N_u$ for Rough NT case if normality condition has to be satisfied. For all other cases, the obtained results matched well with the available results. The results have been interpreted in terms of the failure mechanisms. The total undrained capacity is the algebraic summation of two capacities viz. geotechnical capacity and buoyancy effect. It has been shown that these two capacities are independent of each
other and can be superimposed to get the total capacity for both full tension and no-tension cases. From the results obtained using numerous analyses for soil with undrained shear strength \( s_u \) constant with depth and \( s_u \) proportional to depth, a prediction model has been developed. The transition of failure mechanism is nicely captured by the introduction of factors \( f_{NBA} \) and \( f_{BA} \). A comparison of the proposed design procedure with DNV (2007) guidelines highlights the shortcomings of DNV (2007). Under NT conditions, DNV (2007) overestimates \( V_u \) which may lead to unconservative design. The proposed design steps for predicting the undrained capacity has been presented as a flow chart in Fig. 16.

Analyses were performed considering wide range of parameters, even beyond practical ranges, to establish the extreme limits of the prediction model for soils with uniform \( s_u \) and \( s_u \) proportional to depth. Additional analyses considering a generalised shear strength profile \( s_u = s_{um} + kz \) involving practical values of \( H \), \( \gamma' \), \( s_{um} \) and \( k \) have been carried out. Several combinations of \( s_{um} \) and \( k \) have been chosen to cover all possible values of soil heterogeneity factor \( \kappa \). With these, the robustness of the prediction model have been tested and it has been found that the proposed methodology gives accurate estimation of undrained uplift capacities for buried pipelines with maximum error being less than 8%. The proposed methodology can also be used for obtaining required embedment for buried pipelines corresponding to a design value of uplift capacity \( V_u \). For pipes with intermediate roughness \((0<\alpha<1)\), all parameters can be linearly interpolated based on \( \alpha \) and uplift capacity can be determined for such cases to get reasonably accurate estimation since the fitted parameters do not vary widely with \( \alpha \).

Results are presented in normalized forms which are applicable to any value of parameters as it covers all extreme scenarios. Thus, it can be readily used for comparison with other studies and also in industrial practices. It should be noted that the present study does not consider the effects of strain softening and loading rate on uplift capacity. Based on rate of loading and also on permeability of soil, partial drainage may occur and the capacities under such
scenarios may differ from the results obtained from this study. Nevertheless, the proposed methodology proves to be a significant improvement over current design practices for obtaining undrained uplift capacities of buried offshore pipelines.
A generalised framework to predict undrained uplift capacity of buried offshore pipelines

List of symbols

$\alpha$ Friction coefficient characterising the roughness of pipe-soil interface

$A_s$ Submerged cross-sectional area of pipe

$\beta_1, \beta_2$ Parameters for obtaining $N_u$ under FT and NT conditions respectively

$D$ Diameter of pipe

$D'$ Effective diameter of pipe

$E$ Modulus of elasticity of soil

$\eta$ Empirical factor used for obtaining $V_u$ for FT conditions as per DNV (2007)

$f_{BA}, f_{NBA}$ Factors for obtaining $s_{u, eff}$ for breakaway and no breakaway occurring at interface respectively

$\gamma'$ Submerged unit weight of soil

$H$ Height of soil cover measured from top of pipe

$k$ Rate of increase of $s_u$ with depth

$\kappa$ Soil heterogeneity parameter ($kD/s_{um}$)

$\lambda_1, \lambda_2$ Parameters for obtaining $f_{NBA}$ (Equation 4)

$\lambda_3, \lambda_4$ Parameters for obtaining $f_{BA}$ (Equation 11)

$\nu$ Poisson’s ratio of soil

$N_u$ Undrained uplift capacity factor

$N_u(0)$ $N_u(weights)$ under FT conditions for pipe placed on the surface of seabed

$N_u(weights)$ $N_u$ in weightless soil

$N_u(weights)$ $N_u$ in soil with non-zero submerged unit weight

$s_u$ Undrained shear strength of soil

$s_{um}$ $s_u$ at mudline of seabed
A generalised framework to predict undrained uplift capacity of buried offshore pipelines

\[ s_u(centre) \quad s_u \text{ at level of centre of pipe} \]

\[ s_u(invert) \quad s_u \text{ at level of invert of pipe} \]

\[ s_u,eff \quad \text{Average } s_u \text{ over zone of shear} \]

\[ T \quad \text{Pipe-soil interface tensile capacity} \]

\[ u_{pipe} \quad \text{Displacement of pipe} \]

\[ u_{soil} \quad \text{Displacement of soil} \]

\[ V_u \quad \text{Undrained uplift capacity of buried pipeline} \]

\[ w \quad \text{Embedment depth of pipe invert} \]

\[ w_{limit} \quad w \text{ at which } N_u \text{ reaches its limiting value} \]

\[ W \quad \text{Work done by pipe in displacing the soil above it} \]

\[ W_{soil} \quad \text{Submerged weight of soil column over buried pipeline} \]

\[ \xi_1, \xi_2 \quad \text{Parameters for obtaining } w_{limit} \text{ (Eq. 5)} \]

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A generalised framework to predict undrained uplift capacity of buried offshore pipelines


A generalised framework to predict undrained uplift capacity of buried offshore pipelines

List of Tables

Table 1 Different input parameters for \( s_u = s_{um} (k = 0) \) and \( s_u = kz (s_{um} = 0) \)

Table 2 \( N_u \) for on-bottom pipelines under full tension conditions

Table 3 Values of various parameters used in the prediction model

Table 4 Input parameter combinations for different cases in soils with generalised undrained shear strength profile \( (s_u = s_{um} + kz) \)

Figure Captions

Fig. 1 Problem geometry and undrained shear strength profiles

Fig. 2 Typical mesh details showing boundary conditions and model dimensions \((H/D = 3)\)

Fig. 3 Validation of adopted numerical methodology (a) Comparison of SSFE results with earlier analytical and experimental studies, (b) Comparison of SSFE and LDFE results for smooth pipes in weightless soil and \( T=0 \), (c) Comparison of SSFE and LDFE results for rough pipes in weightless soil and \( T=0 \)

Fig. 4 Undrained uplift capacity factors \((N_u)\) in soils with uniform \( s_u \) under NT and FT conditions for (a) smooth and (b) rough pipes

Fig. 5 Typical normalised displacement \((u_{soil}/u_{pipe})\) contour diagrams during uplift of rough pipes in soil with uniform \( s_u \)

Fig. 6 \( V_u/s_{u(centre)}D \) under FT conditions in soils with \( s_u = kz \) for (a) smooth and (b) rough pipes
A generalised framework to predict undrained uplift capacity of buried offshore pipelines

Fig. 7 Typical normalised displacement \( u_{soil}/u_{pipe} \) contour diagrams during uplift of smooth pipes in soils with \( s_u = k_z \)

Fig. 8 \( V_u/s_{u(centre)}D \) for smooth pipes under NT conditions in soils with \( s_u = k_z \)

Fig. 9 \( V_u/s_{u(centre)}D \) for rough pipes under NT conditions in soils with \( s_u = k_z \)

Fig. 10 Normalised undrained uplift capacities for weightless soils obtained by deducting the effect of buoyancy

Fig. 11 Variation of factors \( f_{NBA} \) and \( f_{BA} \) with soil cover height ratio \( (H/D) \)

Fig. 12 Limiting depth ratios \( (w_{limit}/D) \) for smooth and rough pipes

Fig. 13 Undrained uplift capacity mechanism during breakaway

Fig. 14 Comparison of obtained undrained uplift capacities with that of model predictions for soils with (a) uniform \( s_u \), (b) \( s_u = k_z \) and (c) soils with generalised shear strength profile \( (s_u = s_{um} + k_z) \)

Fig. 15 Comparison of obtained undrained uplift capacities with DNV (2007)

Fig. 16 Proposed design steps for predicting the undrained uplift capacity for buried offshore pipelines
Table 1 Different input parameters for \( s_u = s_{um} \) \((k = 0)\) and \( s_u = k_z \) \((s_{um} = 0)\)

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Undrained shear Strength Profile</th>
<th>( s_u = k_z )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Uniform ( s_u ) along depth</td>
<td></td>
</tr>
<tr>
<td>( \gamma'D/s_u(centre) )</td>
<td>0, 0.1, 0.2, 0.3, 0.4, 0.5, 0.75, 1, 2, 3, 4, 5</td>
<td>-</td>
</tr>
<tr>
<td>( \gamma'/k )</td>
<td>-</td>
<td>0, 1, 2, 5, 10, 20, 50</td>
</tr>
<tr>
<td>( H/D )</td>
<td>0, 0.5, 1, 1.5, 2, 3, 4, 5, 7, 9, 11, 14, 19</td>
<td></td>
</tr>
<tr>
<td>( E/s_u )</td>
<td>500</td>
<td></td>
</tr>
<tr>
<td>( \nu )</td>
<td>0.499</td>
<td></td>
</tr>
<tr>
<td>( \alpha )</td>
<td>0 (for smooth) and 1 (for rough)</td>
<td></td>
</tr>
<tr>
<td>Interface tensile capacity</td>
<td>Zero (for NT) and infinite (for FT)</td>
<td></td>
</tr>
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</table>
Table 2 \( N_u(\text{weightless}) \) for on-bottom pipelines under full tension conditions

<table>
<thead>
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<th>( w/D )</th>
<th>( N_u(\text{weightless}) = V_u / s_u D' )</th>
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<tr>
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<td>Smooth Pipes</td>
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<td>0.1</td>
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<td>0.5</td>
<td>4.75</td>
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<td>1.0</td>
<td>6.18</td>
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Table 3 Values of various parameters used in the prediction model

<table>
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<th>Parameters</th>
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<th>Rough Pipes</th>
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<td>$N_u(\text{limit})$</td>
<td>9.2</td>
<td>12</td>
</tr>
<tr>
<td>$N_u0$</td>
<td>4.5</td>
<td>5</td>
</tr>
<tr>
<td>$\beta_1$</td>
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<td>$\beta_2$</td>
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<td>$\lambda_1$</td>
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<td>$\lambda_3$</td>
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<td>$\lambda_4$</td>
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<td>0.3</td>
</tr>
<tr>
<td>$\xi_1$</td>
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<td>8</td>
</tr>
<tr>
<td>$\xi_2$</td>
<td>0.65</td>
<td>0.85</td>
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</table>
Table 4 Input parameter combinations for different cases in soils with generalised undrained shear strength profile ($s_u = s_{um} + k_z$)

<table>
<thead>
<tr>
<th>Case</th>
<th>$H/D$</th>
<th>$s_{um}$ (kPa)</th>
<th>$k$ (kPa/m)</th>
<th>$kD/s_{um}$</th>
<th>$\gamma'$ (kN/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1: Smooth NT - Perfectly smooth pipe and no tension at pipe-soil interface</td>
<td>1, 4, 9</td>
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<td>0</td>
<td>0.0</td>
<td>2, 5, 8</td>
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<td>5</td>
<td>5.0</td>
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<td>0.8</td>
<td>10</td>
<td>12.5</td>
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<td>0.2</td>
<td>20</td>
<td>50.0</td>
<td>2, 5, 8</td>
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<tr>
<td>Case 2: Rough NT – Fully rough pipe and no tension at pipe-soil interface</td>
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<td>0</td>
<td>0.0</td>
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<td>Case 3: Smooth FT – Perfectly smooth pipe and infinite tension at pipe-soil interface</td>
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<td>0.2</td>
<td>20</td>
<td>50.0</td>
<td>2, 5, 8</td>
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</table>
Profile 1:
Uniform soil, i.e.,
\( k = 0, s_u = s_{um} \)

Profile 2:
Linearly varying with zero mudline shear strength, i.e.,
\( s_{um} = 0, s_u = k z \)

Profile 3:
Generalised shear strength profile, i.e.,
\( s_{um} \geq 0, k \geq 0, s_u = s_{um} + k z \)
Fig. 2 Typical mesh details showing boundary conditions and model dimensions (H/D = 3)

91x95mm (600 x 600 DPI)
Fig. 3 Validation of adopted numerical methodology (a) Comparison of SSFE results with earlier analytical and experimental studies, (b) Comparison of SSFE and LDFE results for smooth pipes in weightless soil and $T=0$, (c) Comparison of SSFE and LDFE results for rough pipes in weightless soil and $T=0$.

(a) 

PRESENT STUDY

- Smooth NT, $\gamma'D/s_u=0$
- Rough NT, $\gamma'D/s_u=0$
- Smooth FT, $\gamma'D/s_u=0$
- Rough FT, $\gamma'D/s_u=0$
- Smooth FT, $\gamma'D/s_u=0.95$
- Rough FT, $\gamma'D/s_u=0.95$

PREVIOUS STUDIES

- Smooth NT, $\gamma'D/s_u=0$
- Rough NT, $\gamma'D/s_u=0$
- Smooth FT, $\gamma'D/s_u=0$
- Rough FT, $\gamma'D/s_u=0$
- Smooth FT, $\gamma'D/s_u=0.95$
- Rough FT, $\gamma'D/s_u=0.95$

$N_{u(limit)}= 9.14$ and $11.94$ for smooth and rough pipes (Randolph and Houlsby, 1984)

(b) 

SSFE, Smooth
- LDFE, Smooth, $H/D = 1$
- LDFE, Smooth, $H/D = 3$
- LDFE, Smooth, $H/D = 5$

(c) 

SSFE, Rough
- LDFE, Rough, $H/D = 1$
- LDFE, Rough, $H/D = 3$
- LDFE, Rough, $H/D = 5$
**Fig. 4** Undrained uplift capacity factors ($N_u$) in soils with uniform $s_u$ under NT and FT conditions for (a) smooth and (b) rough pipes.
Fig. 5 Typical normalised displacement ($\frac{u_{\text{soil}}}{u_{\text{pipe}}}$) contour diagrams during uplift of rough pipes in soil with uniform $s_u$. 

Fig. 5 
296x483mm (300 x 300 DPI)
Fig. 6  $V_u/s_u(centre)D$ under FT conditions in soils with $s_u = kz$ for (a) smooth and (b) rough pipes.
Fig. 7 Typical normalised displacement ($u_{soil}/u_{pipe}$) contour diagrams during uplift of rough pipes in soil with $s_u = kz$

Fig. 7

296x483mm (300 x 300 DPI)
Fig. 8 $V_u/s_u(centre)D$ for smooth pipes under NT conditions in soils with $s_u = kz$
**Fig. 9** $V_u/s_u(centre)D$ for rough pipes under NT conditions in soils with $s_u = k_z$

(a) 

(b)
Fig. 10 Normalised undrained uplift capacities for weightless soils obtained by deducting the effect of buoyancy

(a) Soils with uniform $s_u$

(b) Soils with $s_u = k_z$
Fig. 11 Variation of factors $f_{NBA}$ and $f_{BA}$ with soil cover height ratio ($H/D$)
Fig. 12 Limiting depth ratios ($w_{\text{limit}}/D$) for smooth and rough pipes

![Graph showing limiting depth ratios for smooth and rough pipes](image-url)
Fig. 13 Undrained uplift capacity mechanism during breakaway
Fig. 14 Comparison of obtained undrained uplift capacities with that of model predictions for soils with (a) uniform $s_u$, (b) $s_u = k z$ and (c) soils with generalised shear strength profile ($s_u = s_{um} + k z$)
Fig. 15 Comparison of obtained undrained uplift capacities with DNV (2007)

(a) Soils with uniform $s_u$

(b) Soils with $s_u = kz$

[Graphs showing undrained uplift capacities for different conditions and soil cover height ratios ($H/D$).]

- **DNV**
  - NT, $\alpha = \text{any}, \gamma D/s_u = 0$
  - NT, $\alpha = 0, \gamma D/s_u = 1$
  - FT, $\alpha = 0, \gamma D/s_u = \text{any}$
  - FT, $\alpha = 1, \gamma D/s_u = 0$

- **SSFE**
  - NT, $\alpha = \text{any}, \gamma'/k = 0$
  - NT, $\alpha = \text{any}, \gamma'/k = 1$
  - FT, $\alpha = 0, \gamma'/k = \text{any}$
  - FT, $\alpha = 1, \gamma'/k = 0$
Fig. 16 Proposed design steps for predicting the undrained uplift capacity for buried offshore pipelines

Undrained uplift capacity ($V_u$) prediction methodology for buried offshore pipelines

Under FT ($T = \infty$) Conditions

If $H + D \geq w_{\text{limit}}$

$$f_{\text{safe}} = \frac{1}{1-\exp\left(-\lambda_1 \left(\frac{H}{D}\right)^{\frac{\gamma'}{2}}\right)}$$

$w_{\text{limit}} = \xi_1 \left(\frac{\gamma' D}{s_{\text{limit}}}ight)^{\xi_2}$

Parameters

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If $H + D < w_{\text{limit}}$

$$f_{\text{safe}} = 1-\exp\left(-\lambda_1 \left(\frac{1+H}{D}\right)^{\frac{\gamma'}{2}}\right)$$

$N = N_{\text{kinetic}} + \left[N_{\text{kinetic}} - N_{\text{st}}\right] \left[1-\exp\left(-0.4\left(1+H/D\right)^{\xi_1}\right)\right] - \left(\frac{\gamma'}{s_{\text{eff}} D}\right)^{\xi_2}$

$V_u = N_{\text{st}} s_{\text{eff}} D$

$V_u = N_{\text{st}} s_{\text{eff}} D$

$V_u = N_{\text{st}} s_{\text{eff}} D$

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