A generalised framework to predict undrained uplift capacity of buried offshore pipelines
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Shubhrajit MAITRA
Research Scholar
Department of Civil Engineering
Indian Institute of Technology Bombay
Powai, Mumbai – 400 076
India
Email: shubhrajit.maitra@iitb.ac.in

Santiram CHATTERJEE (corresponding author)
Assistant Professor
Department of Civil Engineering
Indian Institute of Technology Bombay
Powai, Mumbai – 400 076
India
Telephone: +91 – 22 – 2576 5327
Email: sc@civil.iitb.ac.in

Deepankar CHOUDHURY
Professor
Department of Civil Engineering
Indian Institute of Technology Bombay
Powai, Mumbai – 400 076
India
Email: dc@civil.iitb.ac.in

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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.

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1 Introduction

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

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

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1 (Aubeny et al. 2005; Newson et al. 2006; Thusyanthan et al. 2011). Various experimental
 2 works using physical model tests either at normal gravity (Schaminee et al. 1990; Palmer et
 3 al. 2003; El-Gharbawy 2006; Liu et al. 2013; Liu et al. 2015; Chen et al. 2016) or under
 4 enhanced gravity conditions in geotechnical centrifuge (Bransby et al. 2002; Palmer et al.
 5 2003; Cheuk et al. 2007; Thusyanthan et al. 2008) have also been performed to study the
 6 response of soil during pipe uplift. Empirical relations proposed from most of these works are
 7 case specific and valid strictly within the domain in which they were carried out. During
 8 upheaval, the soil response may be drained or undrained depending on permeability of the
 9 soil and the rate of loading. Martin and White (2012) highlighted the shortcomings of DNV
 10 RP-F110 (DNV 2007), which is in vogue in current industrial practice for predicting
 11 undrained uplift capacity relevant for fine grained soils. Previous researchers have pointed
 12 out that uplift failure mechanism undergo a transition from 'global failure mode' to 'local
 13 failure mode' with increase in pipe embedment (Newson *et al.* 2006; El-Gharbawy 2006;
 14 Martin and White 2012). In global failure mode, the mechanism reaches the seabed surface
 15 whereas, in local failure mode, a localised circumferential flow of soil occurs around the pipe
 16 during uplift. However, a proper guideline on the type of mechanism a pipe is likely to
 17 undergo during failure is missing in literature. A generalised framework or prediction model
 18 for estimating undrained uplift capacity of buried pipelines for various shear strength profiles,
 19 pipe embedment, unit weight of soil, soil heterogeneity factor, pipe-soil interface roughness
 20 and tensile capacity also does not exist in literature.

21 In this paper, a comprehensive study has been carried out to estimate undrained uplift
 22 capacity of buried pipelines using finite element software ABAQUS 6.13 (Dassault Systèmes
 23 2013). Analyses were performed for several values of soil cover height ratio (H/D),
 24 normalised submerged unit weight of soil ($\gamma' D/s_u$ for soils with uniform undrained shear
 25 strength (s_u) and γ'/k for soils with s_u proportional to depth), non-dimensional soil

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1 heterogeneity factor ($\kappa=kD/s_{um}$) and extreme values of pipe-soil interface roughness
2 coefficient (α) and interface tensile capacity (T). Fig. 1 shows a schematic representation of
3 the problem geometry and shear strength profiles considered in the present study. A simple
4 but generalised framework that incorporates the effects of all these parameters is proposed for
5 prediction of undrained upheaval resistance of soil. All the results are presented in non-
6 dimensional form so that they can be implemented in design practice.

7 **Finite Element Methodology**

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

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

22 ***Details of analyses***

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1 A detailed parametric study by varying the influencing parameters in a systematic manner
 2 was carried out to understand the undrained uplift behaviour of buried pipelines. Total stress
 3 analyses assuming Poisson's ratio (ν) of 0.499 (≈ 0.50), replicating essentially the constant
 4 volume condition, were performed for soils with s_u constant ($s_u = s_{um}$, $k=0$) and proportional to
 5 depth ($s_{um}=0$, $s_u=kz$). However, in order to overcome numerical limitations, a very small
 6 value of s_{um} ($=0.001\text{kPa}$) was assigned for soils with $s_u=kz$. Modulus of elasticity of soil (E) at
 7 a particular embedment was considered as 500 times of s_u at that location (i.e., stiffness
 8 index, $E/s_u=500$). Undrained uplift capacity is independent of E/s_u for elastic perfectly plastic
 9 constitutive relationship of soil. However, the pipe displacement at which this capacity gets
 10 mobilised is a function of E/s_u . In order to study the effect of submerged unit weight of soil
 11 (γ') on failure mechanisms, several values of γ' (uniform along depth) were considered. Pipe
 12 was assumed to be rigid and weightless. Hence, actual undrained uplift capacity is obtained
 13 by adding pipe weight to the results obtained from present analyses. Pipes are generally made
 14 of steel and are at times coated externally with thermally insulating and anti-corrosive agents.
 15 Depending on this and also on soil particle size, the pipe-soil interface can vary from
 16 'perfectly smooth' to 'fully rough'. Roughness of pipe-soil interface is characterised by
 17 friction coefficient α which is the ratio of mobilised shear strength at the interface to the shear
 18 strength of surrounding soil. Interface was assumed to be either smooth ($\alpha=0$) or fully rough
 19 ($\alpha=1$). During uplift process, a gap may get formed below pipe if adequate interface tensile
 20 strength (T) is not mobilised. Extreme values of T were considered in the present study ($T=0$
 21 corresponds to no tension (NT) while $T=\infty$ corresponds to full tension (FT) mobilised at
 22 interface). Thus, the following four cases were considered: (i) Smooth FT: Smooth pipe with
 23 unlimited tensile capacity at interface, (ii) Smooth NT: Smooth pipe with zero interface
 24 tension, (iii) Rough FT: Rough pipe with infinite interface tensile strength, and (iv) Rough
 25 NT: Rough pipe with no tension developed at interface. To model Rough FT conditions, the

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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

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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

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

14 **Results and discussions**

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

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

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

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

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

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

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

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

19 The embedment at which N_u reaches its limiting value (w_{limit}) depends on $\gamma'D/s_{u(centre)}$ i.e., on
 20 $(\gamma'/k)(D/(w-0.5D))$ and α . Hence, analyses were performed by varying the non-dimensional
 21 term γ'/k over wide ranges for both smooth and rough pipes. Fig. 6 shows the obtained V_u
 22 normalised with respect to $s_{u(centre)} D$ for smooth and rough pipes under FT conditions in soils
 23 with $s_u = kz$. Few examples of uplift mechanism for smooth pipes are depicted in Fig. 7. From

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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 $(9.2-\gamma'A_s/s_{u(centre)}D)$ and $(12-\gamma'A_s/s_{u(centre)}D)$ 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 $\gamma'A_s/s_{u(centre)}D$. For the cases where limiting resistance is not mobilised (e.g. $\gamma'/k=0$, $H/D \leq 2$ 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. $\gamma'/k=0$, $H/D=0$). 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. $\gamma'/k=10$, $H/D=0$) 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 , $\gamma'D/s_{u(centre)}$ 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

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1 buoyancy effect, N_u decrease due to substantial increase in $\gamma'A_s/s_{u(\text{centre})}D$ at shallow
 2 embedment. However, with increase in embedment, $\gamma'A_s/s_{u(\text{centre})}D$ decreases which results in
 3 an increase in N_u . Normalisation of uplift capacity using $s_{u(\text{centre})}D$ is not justifiable for all the
 4 cases where breakaway occurs. Under such circumstances soil lying above the pipe is
 5 displaced and the average s_u of the displaced soil is lower than $s_{u(\text{centre})}$. All these factors
 6 highlight the complexity of the problem for soils with $s_u=kz$.

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

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

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

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

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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 γ'/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

$$(1) \quad N_u = \frac{V_u}{s_{u,eff} D} = N_{u(weightless)} - \left(\frac{\gamma'}{s_{u,eff} D} \right) A_s$$

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

$$(2) \quad N_u = \frac{V_u}{s_{u,eff} D} = N_{u0} + (N_{u(limit)} - N_{u0}) \left[1 - \exp \left\{ -0.4 \left(1 + H/D \right)^{\beta_1} \right\} \right] - \left(\frac{\gamma'}{s_{u,eff} D} \right) A_s$$

Here, $N_{u(limit)}$ are the limiting values of $N_{u(weightless)}$ as proposed by Randolph and Houlsby (1984) and β_1 is a curve fitting parameter (see Table 3). For soils with uniform s_u , $s_{u,eff} = s_u$, whereas for soils with $s_u = kz$, $s_{u,eff}$ is defined by

$$(3) \quad s_{u,eff} = f_{NBA} \cdot k \cdot \left(H + \frac{D}{2} \right)$$

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Here, f_{NBA} is a factor for obtaining $s_{u,eff}$ for the 'kz' 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 $s_u=kz$ and uniform s_u . Finally, f_{NBA} is expressed using eq. (4) with λ_1 and λ_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 $s_u=kz$, $s_{u,eff}$ 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} .

14 *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 α 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.

$$(5) \quad \left(\frac{w_{limit}}{D} \right) = \xi_1 \times \left(\frac{\gamma' D}{s_{u,centre}} \right)^{-\xi_2}$$

The values of curve fitting parameters ξ_1 and ξ_2 presented in Table 3.

No Tension (zero interface tensile capacity)

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

For $w \geq w_{limit}$, 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 $w < w_{limit}$, 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 (ΔV_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 (δW) done by the elemental pipe segment in displacing the soil above it by δu is

$$(6) \quad \delta W = \gamma' \left(H + \frac{D}{2} - \frac{D}{2} \sin \theta \right) \times \left(\frac{D}{2} \sin \theta d\theta \right) \times \delta u$$

Thus, work done during uplift can be equated with ΔV_u as follows

$$(7) \quad \Delta V_u \times \delta u = 2 \int_0^{\pi/2} \left[\gamma' \left(H + \frac{D}{2} - \frac{D}{2} \sin \theta \right) \times \left(\frac{D}{2} \sin \theta \right) \right] \delta u d\theta$$

Equation 7 can be simplified to obtain ΔV_u and is expressed as

$$(8) \quad \Delta V_u = \gamma' \left[HD + D^2 \left(\frac{1}{2} - \frac{\pi}{8} \right) \right] = W_{soil}$$

Hence, it can be observed from eq. (8) that ΔV_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 γ'/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

$$(9) \quad N_{u(weighty)} = N_{u(weightless)} + \frac{W_{soil}}{s_{u,eff}D}$$

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

$$(10) \quad N_u = \frac{V_u}{s_{u,eff}D} = N_{u(limit)} \left(1 - \frac{1}{1 + 0.2 \left(\frac{H}{D} + 0.5 \right)^{\beta_2}} \right) + \frac{W_{soil}}{s_{u,eff}D}$$

Here, β_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

$$(11) \quad s_{u,eff} = f_{BA} \cdot k \cdot \left(H + \frac{D}{2} \right)$$

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_{NBA} and is expressed as

$$(12) \quad f_{BA} = 1 - \exp \left(-\lambda_3 \left(1 + \frac{H}{D} \right)^{\lambda_4} \right)$$

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1 Here, curve fitting parameters λ_3 and λ_4 are listed in Table 3. When breakaway occurs, soil
2 above the level of pipe centre gets displaced. Thus, for $s_u=kz$, $s_{u(centre)} \geq s_{u,eff}$ and $f_{BA} \leq 1$. At
3 infinite embedment, $f_{BA}=1$ as w_{limit} for weightless soil is also infinity.
4 The obtained undrained uplift capacities ($V_{u(obtained)}$) using ABAQUS are compared with those
5 predicted by the proposed model ($V_{u(predicted)}$) for soils with uniform s_u and $s_u=kz$ and are
6 represented in Fig. 14(a) and 14(b). It is observed that the error is within 10% for most cases.
7 For few cases with very shallow embedment and lower soil unit weight, the errors exceeded
8 10% as the capacity is very low and small error involved in curve fitting magnifies the %
9 error. Nevertheless, for all sets of practically encountered values of parameters, margin of
10 error is quite low which is dealt with in the next section.

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

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

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

$$20 \quad (13) \quad s_{u,eff} = s_{um} + f_{NBA} \cdot k \cdot \left(H + \frac{D}{2} \right)$$

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

$$22 \quad (14) \quad s_{u,eff} = s_{um} + f_{BA} \cdot k \cdot \left(H + \frac{D}{2} \right)$$

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1 For these set of analyses, values of H/D , s_{um} and k were chosen over wide ranges which are
 2 often met with in reality. Values of k and s_{um} were taken considering large variation of soil
 3 heterogeneity factor (κ) often observed in offshore soil conditions. Table 4 shows the
 4 combinations of input parameters considered for generalised undrained shear strength profile
 5 of soil. Undrained uplift capacity were calculated for all combinations ($V_{u(predicted)}$) using the
 6 proposed model (eq. (2), (4), (5), (8), (10), (12) – (14)). Fig. 14(c) shows the comparison of
 7 $V_{u(predicted)}$ with the obtained values ($V_{u(obtained)}$) using ABAQUS. Results show that error for
 8 the combinations considered is <5% for most of the cases whereas, the maximum error is
 9 within 8%. Thus, the proposed undrained uplift capacity prediction model gives correct and
 10 critical results and hence can be adopted in design practice.

11 *Comparison of proposed methodology with DNV (2007) guidelines*

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

- 15 (i) DNV (2007) suggests that undrained uplift capacities are governed by two main
 16 failure mode: “Local soil failure” and “Global soil failure”. As per DNV (2007), local soil
 17 failure mode corresponds to FT conditions, whereas soil fails globally when breakaway
 18 occurs under NT conditions. However, it does not state that even under NT conditions,
 19 breakaway may not occur (i.e., when $w > w_{limit}$) and a transition in failure mechanism takes
 20 place with increase in H/D .
- 21 (ii) DNV (2007) assumes a simple vertical slip model for global soil failure mode. Thus,
 22 when breakaway occurs, capacities become independent of α . This assumption is not realistic
 23 as slip surfaces may not be vertical (as evident from Figs. 5 (c) – (e) and 7 (c) – (d)). The

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- 1 length of the failure surface varies with α and thus the capacities are more for rough
 2 interfaces (upto 30%) as compared to smooth interfaces.
- 3 (iii) As also pointed out by Martin and White (2012), vertical slip model overestimates V_u
 4 (see Fig. 15(a) and 15(b)) and is thus unconservative.
- 5 (iv) Under FT conditions, DNV (2007) does not consider the detrimental effect of soil
 6 buoyancy on V_u . Moreover, it recommends use of an empirical factor η (which lies in the
 7 range 0.55-0.8 and is typically taken as 0.65) for obtaining V_u . The use of this factor
 8 underestimates V_u (upto 35%) in soils with low $\gamma'D/s_{u(\text{centre})}$.
- 9 (v) DNV (2007) suggests that limiting capacities are mobilized under FT conditions at
 10 $H/D = 4$. However, from present study it is evident that limiting capacities are attained at H/D
 11 roughly equal to 1.2 and 2.2 for smooth and rough pipes respectively.

12 Conclusions

13 The present study deals with development of a simple unified prediction model for undrained
 14 uplift capacities of buried pipes in soil. Analyses were carried out for the following four
 15 cases: a) Smooth NT, b) Rough NT, c) Smooth FT and d) Rough FT using finite element
 16 based software ABAQUS. Additional LDFE analyses carried out for few cases indicated that
 17 the SSFE model was able to capture peak uplift resistances though the problem involves large
 18 displacements. The results were validated with presently available theoretical and
 19 experimental studies in the literature. From the present study, it has been found that for
 20 $w < w_{\text{limit}}$, plasticity theory solutions overestimate N_u for Rough NT case if normality condition
 21 has to be satisfied. For all other cases, the obtained results matched well with the available
 22 results. The results have been interpreted in terms of the failure mechanisms. The total
 23 undrained capacity is the algebraic summation of two capacities viz. geotechnical capacity
 24 and buoyancy effect. It has been shown that these two capacities are independent of each

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1 other and can be superimposed to get the total capacity for both full tension and no-tension
 2 cases. From the results obtained using numerous analyses for soil with undrained shear
 3 strength (s_u) constant with depth and s_u proportional to depth, a prediction model has been
 4 developed. The transition of failure mechanism is nicely captured by the introduction of
 5 factors f_{NBA} and f_{BA} . A comparison of the proposed design procedure with DNV (2007)
 6 guidelines highlights the shortcomings of DNV (2007).. Under NT conditions, DNV (2007)
 7 overestimates V_u which may lead to unconservative design. The proposed design steps for
 8 predicting the undrained capacity has been presented as a flow chart in Fig. 16.

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

21 Results are presented in normalized forms which are applicable to any value of parameters as
 22 it covers all extreme scenarios. Thus, it can be readily used for comparison with other studies
 23 and also in industrial practices. It should be noted that the present study does not consider the
 24 effects of strain softening and loading rate on uplift capacity. Based on rate of loading and
 25 also on permeability of soil, partial drainage may occur and the capacities under such

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- 1 scenarios may differ from the results obtained from this study. Nevertheless, the proposed
- 2 methodology proves to be a significant improvement over current design practices for
- 3 obtaining undrained uplift capacities of buried offshore pipelines.

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List of symbols

α	Friction coefficient characterising the roughness of pipe-soil interface
A_s	Submerged cross-sectional area of pipe
β_1, β_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
η	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
γ'	Submerged unit weight of soil
H	Height of soil cover measured from top of pipe
k	Rate of increase of s_u with depth
κ	Soil heterogeneity parameter (kD/s_{um})
λ_1, λ_2	Parameters for obtaining f_{NBA} (Equation 4)
λ_3, λ_4	Parameters for obtaining f_{BA} (Equation 11)
ν	Poisson's ratio of soil
N_u	Undrained uplift capacity factor
N_{u0}	$N_{u(weightless)}$ under FT conditions for pipe placed on the surface of seabed
$N_{u(weightless)}$	N_u in weightless soil
$N_{u(weighty)}$	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

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$s_{u(\text{centre})}$	s_u at level of centre of pipe
$s_{u(\text{invert})}$	s_u at level of invert of pipe
$s_{u,\text{eff}}$	Average s_u over zone of shear
T	Pipe-soil interface tensile capacity
u_{pipe}	Displacement of pipe
u_{soil}	Displacement of soil
V_u	Undrained uplift capacity of buried pipeline
w	Embedment depth of pipe invert
w_{limit}	w at which N_u reaches its limiting value
W	Work done by pipe in displacing the soil above it
W_{soil}	Submerged weight of soil column over buried pipeline
ζ_1, ζ_2	Parameters for obtaining w_{limit} (Eq. 5)

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

Parameters	Undrained shear Strength Profile	
	Uniform s_u along depth	$s_u = kz$
$\gamma'D/s_{u(\text{centre})}$	0, 0.1, 0.2, 0.3, 0.4, 0.5, 0.75, 1, 2, 3, 4, 5	-
γ'/k	-	0, 1, 2, 5, 10, 20, 50
H/D	0, 0.5, 1, 1.5, 2, 3, 4, 5, 7, 9, 11, 14, 19	
E/s_u	500	
ν	0.499	
α	0 (for smooth) and 1 (for rough)	
Interface tensile capacity	Zero (for NT) and infinite (for FT)	

Table 2 $N_{u(weightless)}$ for on-bottom pipelines under full tension conditions

w/D	$N_{u(weightless)} = V_u / s_u D'$	
	Smooth Pipes	Rough Pipes
0.01	4.50	5.00
0.1	4.53	5.10
0.5	4.75	5.66
1.0	6.18	7.47

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Table 3 Values of various parameters used in the prediction model

Parameters	Smooth Pipes	Rough Pipes
$N_{u(limit)}$	9.2	12
N_{u0}	4.5	5
β_1	3	2
β_2	1.3	1
λ_1	1	0.85
λ_2	2.2	2
λ_3	0.55	0.60
λ_4	0.4	0.3
ζ_1	5	8
ζ_2	0.65	0.85

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

Case	H/D	s_{um} (kPa)	k (kPa/m)	kD/s_{um}	γ' (kN/m ³)
Case 1: Smooth NT - Perfectly smooth pipe and no tension at pipe- soil interface	1, 4, 9	20	0	0.0	2, 5, 8
		10	2	0.2	2, 5, 8
		2	2	1.0	2, 5, 8
		1	5	5.0	2, 5, 8
		0.8	10	12.5	2, 5, 8
		0.2	20	50.0	2, 5, 8
Case 2: Rough NT – Fully rough pipe and no tension at pipe-soil interface	1, 4, 9	20	0	0.0	2, 5, 8
		10	2	0.2	2, 5, 8
		2	2	1.0	2, 5, 8
		1	5	5.0	2, 5, 8
		0.8	10	12.5	2, 5, 8
		0.2	20	50.0	2, 5, 8
Case 3: Smooth FT – Perfectly smooth pipe and infinite tension at pipe-soil interface	1, 4, 9	20	0	0.0	2, 5, 8
		10	2	0.2	2, 5, 8
		2	2	1.0	2, 5, 8
		1	5	5.0	2, 5, 8
		0.8	10	12.5	2, 5, 8
		0.2	20	50.0	2, 5, 8
Case 4: Rough FT – Fully rough pipe and infinite tension at pipe- soil interface	1, 4, 9	20	0	0.0	2, 5, 8
		10	2	0.2	2, 5, 8
		2	2	1.0	2, 5, 8
		1	5	5.0	2, 5, 8
		0.8	10	12.5	2, 5, 8
		0.2	20	50.0	2, 5, 8

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91x95mm (600 x 600 DPI)

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Fig. 7 Typical normalised displacement (u_{soil}/u_{pipe}) contour diagrams during uplift of rough pipes in soil with $s_u = kz$
296x483mm (300 x 300 DPI)

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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) $s_u = kz$ and (c) soils with generalised shear strength profile ($s_u = s_{um} + kz$)

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

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Author/s:

Maitra, S;Chatterjee, S;Choudhury, D

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