Numerical modelling of seepage and tension beneath plate anchors

Revised manuscript submitted to Computers and Geotechnics on 21/11/2018

Shubhrajit MAITRA

Research Scholar

Department of Civil Engineering

Indian Institute of Technology Bombay

Powai, Mumbai – 400 076, India

Email: shubhrajit.maitra@iitb.ac.in

David WHITE

Professor

Faculty of Engineering and the Environment

University of Southampton,

Southampton SO17 1BJ, UK

Formerly of Centre for Offshore Foundation Systems

The University of Western Australia

Crawley, Western Australia – 6009, Australia

Email: david.white@soton.ac.uk

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

Institute Chair Professor

Department of Civil Engineering

Indian Institute of Technology Bombay

Powai, Mumbai – 400 076, India

Email: dc@civil.iitb.ac.in

No. of words: 6845

No. of tables: 1

No. of figures: 14

Numerical modelling of seepage and tension beneath plate anchors

Shubhrajit Maitra, David White, Santiram Chatterjee and Deepankar Choudhury

Abstract

The uplift capacity of buried plate anchors depends on the tension sustained beneath the anchor. This study provides a detailed treatment of interface tension and the associated seepage flow and gap formation below the anchor. This behaviour is explored via large deformation finite element analysis using a thin highly compressible porous layer for the gap. The observed seepage effect is captured by a simple model using Hvorslev's intake factor, validated across a wide parameter range. This model for uplift capacity at various pull-out rates provides a simple basis for the effect of seepage on plate anchor capacity under sustained loading.

Keywords: Anchors; finite element modelling; offshore engineering; seepage; soil-structure interaction.

Words: 6845, **Tables:** 1, **Figures:** 14.

1. Introduction

Offshore oil and gas exploration facilities are expanding from near-shore locations to deeper water regions to fulfil global energy requirements. These developments in great water depths have led to the evolution of offshore foundation concepts from fixed-base platforms at shallow water to buoyant facilities at deep water. These buoyant structures are kept in place by mooring them to the seabed using anchoring systems and thus, anchors are subjected to large pull-out loads. Several types of anchors have been adopted in sites across globe such as gravity anchors, anchor piles, suction caissons, traditional fixed fluke and plate anchors (Randolph and Gourvenec [1]). The plate anchors are often keyed into the seabed at various inclinations based on direction of loading. Deeply buried plate anchors are subjected to uplift forces coming from the attached mooring line and thus precise estimation of uplift resistance is essential for design of these anchors.

The uplift mechanism of buried plate anchors is a complex soil-structure interaction problem involving several factors. These factors can be broadly classified into three categories: (i) structural properties (shape, size and embedment); (ii) soil properties (shear strength profile, submerged unit weight of soil); and (iii) soil-structure interface properties (mobilised interface tension, interface roughness). Several researchers have studied anchor-soil interaction problems using theoretical approaches like classical plasticity theory and limit equilibrium based solutions (Vesic [2]; Rowe [3]; Rao and Kumar [4]; Merifield et al. [5], [6], [7]; Choudhury and Rao [8]; White et al. [9]; Rangari et al. [10]). Results from physical model tests at normal gravity (Vesic [2]; Singh and Ramaswamy [11]) or at enhanced gravity using geotechnical centrifuge (Wang and O'Loughlin [12]; Chen et al. [13]) are also reported in the literature. Numerical modelling has been widely used for studying soil-structure interactions for plate anchors (Yu et al. [14]; Singh et al. [15]; Liu et al. [16]). Liu et al. [16]

considered wide range of aspect ratios and highlighted the effects of plate shape on anchor uplift capacity.

From these studies, the total uplift resistance offered by soil depends on the following three aspects: (a) shearing resistance along slip planes or shear zones; (b) contribution of soil weight towards uplift resistance; and (c) interface tensile resistance (T) that may be mobilised due to generation of negative excess pore pressure below the anchor. The effects of the first two aspects are well documented in literature for buried objects (Yu et al. [14]; Martin and White [17]; Maitra et al. [18]; Singh et al. [15]) whereas, the cause and effect of interface tension has not been studied in detail. Earlier studies on plate anchors (Yu et al. [14]; Singh et al. [15]) are limited to the cases that assume zero or infinite tensile capacities (i.e., “No Tension” (NT) corresponding to $T \approx 0$ and “Full Tension” (FT) for $T \approx \infty$) only. These studies show that the capacity for strip anchors may vary largely with T and thus, studying in more detail the origin and effect of T on uplift capacity becomes important.

The creation of interface tension requires negative excess pore pressure to be mobilised beneath the anchor plate, and such excess pore pressure sets up a tendency for seepage flow associated with the difference in pore pressures between the two sides of the anchor and also the free field during pull-out. During uplift, based on the soil permeability (k) and pull-out velocity (v), a varying level of negative excess pore pressure (measured in comparison to in-situ pore pressure) is generated. Thus, for the cases of soil with low permeability and faster pull-out rate, interface tension can be much higher as compared to slower pull-out in more permeable soils. If adequate interface tension is mobilised during uplift (or if the self-weight of the adjacent soil causes it to collapse and apply upward compression on the underside of the anchor), the soil beneath the anchor is involved in the failure mechanism in addition to the soil above it. This is sometimes referred to as a two-sided mechanism. The level of

1 tension therefore affects the zones of shearing that can be mobilised. On the other hand, for
2 slow pull-out rates, separation at anchor-soil interface (referred to as breakaway) may occur,
3 particularly for anchors placed at shallow embedment where the seepage path is short,
4 resulting in a one-sided mechanism. In such cases a gap filled with water is formed below the
5 anchor, and this has been observed in experiments (Han et al. [19]).
6
7
8
9
10

11
12 Studies have also shown that a transition in mechanism from a global shear mechanism (with
13 slip planes extending to the ground surface) to a local mechanism (with slip planes extending
14 from top to bottom of the anchor) occurs with increase in embedment (Yu et al. [14]; Martin
15 and White [17]; Maitra et al. [18]; Singh et al. [15]). The depth at which this transition of
16 mechanism occurs under NT conditions has been defined as limiting embedment (w_{limit}) by
17 earlier researchers (Maitra et al. [18]).
18
19
20
21
22
23
24
25
26
27

28 Studying the effects of seepage and gap formation on uplift response via numerical modelling
29 is challenging. Attempts have been made to study gap formation and associated passive
30 suction for other foundation systems like suction caissons and skirted foundations (Cao [20];
31 Mana et al. [21]) by using a thin soft porous layer of “gap” or “water” elements beneath the
32 buried structure, but such studies have not been performed for plate anchors. For plate
33 anchors, Han et al. [19] improved on previous plate anchors studies by modelling the
34 initiation of a void beneath the plate (rather than a water-filled gap). This approach may be
35 simpler, but does not then enforce continuity of volume to control the growth of the void and
36 the influence on anchor velocity. Design practices therefore currently consider only the
37 extreme values of T (0 or ∞) which is a severe limitation. The current DNV guideline (DNV
38 [22]) does not address the effect of gap formation on pull-out capacity. It recommends an
39 uplift capacity factor, N_u , varying from 5.14 (at the surface) to 12 (at embedment ratio greater
40
41
42
43
44
45
46
47
48
49
50
51
52
53
54
55
56
57
58
59
60
61
62
63
64
65

than 4.5). However, gap formation can result in much lower capacities at lower embedment levels.

To allow more accurate design of buried anchors, and to capture correctly the influence of seepage on interface tension, the present study introduces a thin layer of “gap” elements below the anchor to numerically simulate gap formation due to inwards seepage flow during sustained pull-out loading. A series of large deformation finite element (LDFE) analyses are performed for pull-out at various rates considering several cases of anchor width (B), embedment (w), soil shear strength (s_u) profiles and buoyant unit weight of soil (γ'). In the later part of the study, strain rate effects on undrained shear strength of soil have also been considered. The current study concludes with a new analytical solution for quantifying the interface tensile forces using a seepage analysis that is analogous to Hvorslev’s intake factors, providing a significant improvement over current design practices.

2. Problem statement

The present study aims at studying the factors that influence uplift capacity for plate anchors. As discussed earlier, v and k are key factors affecting interface tension. The other important factor is the problem geometry (anchor width and embedment) because the location of the anchor with respect to the seabed governs the flow path length and the dissipation of negative excess pore pressure during uplift. In addition, soil weight plays an important role during uplift if the anchor displaces a volume of soil, or if a gap forms beneath it. The limiting embedment ratio, $(w/B)_{\text{limit}}$, decreases with increase in normalised unit weight (Singh et al. [15]). For $(w/B) \geq (w/B)_{\text{limit}}$, the formation of a gap beneath the anchor is not feasible because the self-weight stresses in the soil are sufficient to cause collapse of the soil into the zone beneath the anchor leading to an upwards compression and a positive effective stress on the

underside of the anchor. For such cases the capacity becomes independent of seepage into the gap.

To capture the combined effect of interface tension and self-weight of soil on gap formation and uplift capacity, several values of embedment ratio (w/B) and normalised submerged unit weight of soil ($\gamma'B/s_u$) were considered and the anchor was subjected to pull-out at various normalised velocities (v/k). Figure 1 shows the problem geometry in a simplified manner.

3. Numerical Methodology

3.1 Material model and input parameters

The anchor was assumed to be rigid and weightless. Several values of embedment were considered ranging from B to $10B$, where B is the width of the anchor. The thickness of the anchor (t) was assumed to be $0.1B$.

In this study, the soil is modelled as an elastic-plastic porous material with fluid flow ensuring volume conservation during flow. Soil behaviour is captured using a coupled pore pressure and effective stress analysis and the constitutive behaviour of soil is represented using a yield criterion equivalent to the Tresca yield criterion where the shear strength of the soil is half of the difference between major and minor (total or effective) principal stresses at failure. Carter et al. [23] studied stress and pore pressure changes for cylindrical cavity expansion problem using this constitutive relationship as well as modified Cam-Clay model. From the study it was found that pore pressure dissipation is relatively independent of choice of soil model. However, the Tresca constitutive model does not capture the change in shear strength of soil due to fluid flow and consolidation. An effective stress soil model that includes volume change would capture this change in shear strength which may in turn gradually reduce the anchor pullout velocity. The aim of the present study is restricted to

quantifying the important effect of seepage in isolation on interface tension and to identify the role that an opening gap has on the capacity and movement of the anchor. Thus, the present analysis may be conservative, particularly for slower pull-out, but could also be combined with calculation methods that consider the rise in mobilised shear strength due to sustained loading on foundations and anchors (e.g. Gourvenec et al. [24], Zhou et al. [25]). Since uplift capacity can vary widely between the two extreme values of interface tension (Yu et al. [14]; Martin and White [17]; Maitra et al. [18]; Singh et al. [15]), quantifying uplift behaviour for seepage beneath the anchor and the resulting variation in interface tension is important, and this is the focus of the present study. Also, in the later part of the study, the effect of strain rate on undrained shear strength is considered using the modified Tresca soil model proposed by Einav and Randolph [26] and Zhou and Randolph [27].

For the primary set of analyses, the undrained shear strength (s_u) was assumed to be uniform with depth and rate effects were not considered. Analyses were performed for both weightless soil ($\gamma'B/s_u = 0$) and soil with weight ($\gamma'B/s_u = 1$ and 2). In the later part of this study, various shear strength profiles and rate effect parameters have been considered which will be discussed in the subsequent sections. The deformation modulus (E_{soil}) was set to $500s_u$ combined with a Poisson's ratio of 0.3. Several values of ν and k were considered to study the effect of pull-out rate. It was found that the uplift capacity depended on the ratio ν/k and thus, in most of the analyses, k was assigned a constant value (10^{-7} m/s) and ν/k was varied from 0.01 to 1000 by varying the pull-out velocity.

In order to numerically model the formation of a gap beneath the anchor – referred to as 'breakaway' – it is necessary to provide the opportunity for the soil and the anchor to separate, subject to (i) mass conservation - since water must flow into the gap to create the space, the increase in volume of the gap should be equal to the volume of water inflow into

the gap; and (ii) equilibrium and pressure transfer, such that an equal total stress applies across the gap, but negligible effective stress, to model a volume of water. These requirements are met by a thin soft porous layer of “gap” elements being placed below the anchor (similar to the approach used by Cao [20] and Mana et al. [21]). By virtue of the porosity of these “gap” elements, tension or negative excess pore pressure can be imparted on the base of the plate thereby contributing to uplift resistance. In order to simulate a gap filled with water, the following criteria need to be satisfied:

- (a) Ideally the deformation modulus of “gap” (E_g) and effective stresses within the gap should be zero. $E_g \approx 0$ ensures that this layer could stretch freely (i.e., to incorporate water flow into the gap) without applying any additional resistance to uplift by virtue of its shear stiffness. However, due to numerical constraints to maintain stability, small values of E_g were considered in the range of 0.1-10 kPa. Over this range the calculated pullout resistance was unaffected by the specific value of E_g and 1 kPa was used for all analyses in this paper. In order to ensure zero effective stress condition within the gap, the stresses within the gap were reset to zero after each small incremental displacement of the large deformation finite element analysis. These aspects are discussed in the next section. Poisson’s ratio for the gap element was assumed to be 0.01 (≈ 0), similar to Mana et al. [21], while the pore fluid is incompressible.
- (b) The excess pore pressure within the gap should be uniform at any instant of time. This was ensured by assigning very high permeability to the “gap” layer ($k_g/k = 10^7$).
- (c) The initial thickness of the gap (t_g) should be ideally zero. However, in order to incorporate the seepage flow into the gap, a thin soft porous layer has been used from the beginning of analyses. t_g was varied from $0.01B$ to $0.06B$ and was found to have negligible effect on obtained results. An intermediate value of t_g was assumed ($= 0.04B$).

for all further analyses in order to avoid excessively small-sized elements and also to ensure adequate number of elements within the gap.

3.2 *Finite element methodology*

The large deformation finite element (LDFE) approach involving “Remeshing and Interpolation Technique with Small Strain” (RITSS) (Hu and Randolph [28], [29]) was used in a 2-D plane strain finite element (FE) model using commercial FE package Abaqus (Dassault Systèmes [30]). The FE model adopted in the current study consisted of two different parts: (a) Anchor and (b) Soil domain and gap layer. The latter part was partitioned so that separate material properties could be assigned to the soil and the “gap”. The soil and the gap were discretised using CPE6MP elements available in element library of Abaqus (i.e., six-noded modified plane strain triangular elements with displacement at all nodes and pore pressure at corner nodes as degrees of freedom). The sides of the soil domain were restrained against horizontal displacements and the bottom of the mesh was assigned zero displacement boundary conditions in both horizontal and vertical directions. Zero pore pressure boundary conditions were assigned to the top of the mesh. A typical FE mesh for $w/B = 2$ is shown in Fig. 2. Mesh optimisation for fixing model dimensions and mesh densities at various locations was performed to increase the computational efficiency and ensure minimal effect on the accuracy of the results. Separation was not allowed between any of the interacting surfaces (anchor-soil, anchor-gap and gap-soil) by using the “tie” constraint in Abaqus. Fig. 3 highlights the need for using gap elements for modelling breakaway beneath anchor. The figure shows a comparison of uplift mechanisms in the presence and absence of the gap elements. In the absence of gap elements, separation has to be either allowed (“No Tension” conditions) or prevented (“Full Tension” conditions) at the anchor-soil interface in the definition of the normal behaviour at the interacting surfaces. However, this does not

incorporate a continuity condition. A continuity check ensures that, if a gap forms, sufficient volume of water must seep into the gap to match the increase in gap volume. This condition creates a more realistic effect of pull-out rate on separation and resistance as separation and growth of gap is feasible only if the anchor is pulled at a velocity that allows seepage of water into the enlarging gap below. This aspect is not captured if gap elements are not used as volume conservation during seepage of water into the gap is not maintained in such cases. In Figs. 3(a) and (b) where gap elements have not been used, the mechanisms are independent of pull-out rate. On the other hand, when gap elements are used, the uplift mechanisms are different at various pull-out rates (see Fig. 3(c)). Also, the change in mechanism from “No Tension” to “Full Tension” mechanisms with increase in v/k is captured properly only if gap elements are used (mechanism for $v/k = 0.01$ in Fig. 3(c) is same as mechanism shown in Fig. 3(b); whereas, mechanism for $v/k = 1000$ in Fig. 3(c) matches with that of Fig. 3(a)).

In a typical LDFE analysis, the large deformation is split into a series of small increments and a small strain finite element (SSFE) analysis is performed for each increment. In the present set of analyses, the anchor was subjected to a displacement of 2% of B in each step. After each SSFE analysis, remeshing is performed for the deformed domain in conjunction with mapping of stresses and other field variables from the old mesh to the new mesh. As a part of this mapping, stresses were recovered from Gauss points to the nodes of the old mesh (using superconvergent patch recovery (SPR) technique: Zienkiewicz and Zhu [31]) followed by interpolation of stresses and pore pressures from the old mesh to the new mesh. The mapping of field variables was done using subroutine written in Fortran. All other pre-processing and post-processing including remeshing was carried out using Python which is the inbuilt scripting language of Abaqus. The entire procedure was continued until fully plastic conditions were mobilised within the soil around the anchor, providing the steady or limiting uplift resistance for the modelled conditions.

1 In reality, effective stresses cannot build up in the gap as it consists of water below the anchor
2 and uplift resistance is provided by virtue of negative excess pore pressure only. Thus,
3
4 although pore pressure was mapped for both the soil and “gap” layer, stress mapping was
5
6 carried out for the soil portion only. The effective stresses within the gap were reset to zero
7
8 (termed here as a stress correction) at the beginning of each SSFE step. Fig. 4 highlights the
9
10 necessity of this resetting of stresses, particularly for small v/k , after each small anchor
11
12 displacements. For the cases of slow pull-out rate (here, $v/k = 0.01$), seepage occurs into the
13
14 gap and the gap stretches resulting in large magnitude of strains within these elements. Thus,
15
16 if this correction is not made, the gap elements progressively offer a small but increasing
17
18 level of resistance during uplift (as $E_g \neq 0$ in the numerical model), even after fully plastic
19
20 conditions are developed within the soil (compare markers with lines for $v/k = 0.01$).
21
22
23
24
25
26

27 For very fast pull-out (here, $v/k = 1000$), seepage into the gap is not feasible and thus the gap
28
29 does not stretch. Thus, the uplift resistance (V) is the same with and without this stress
30
31 correction (compare markers with lines for $v/k = 1000$). Another important advantage of this
32
33 stress correction is that the results become virtually insensitive to E_g . Since it is possible to
34
35 apply this correction in LDFE analyses, the present methodology is a significant
36
37 improvement over earlier SSFE methodologies (like Cao [20]; Mana et al. [21]).
38
39
40
41
42

43 ***3.3 Validation of the present methodology***

44
45
46
47 In the present numerical model, the top of the anchor behaves as a fully rough interface
48
49 (because of using “tie” constraint) whereas the bottom behaves as a smooth surface because
50
51 of presence of soft porous medium with negligible shear stiffness. Earlier studies (Yu et al.
52
53 [14]; Singh et al. [15]) have shown that the interface roughness coefficient ($\alpha = 0, 1$ for
54
55 perfectly smooth and rough interface respectively) does not influence capacity significantly
56
57
58
59
60
61
62
63
64
65

for horizontal strip anchors. Thus, the results can be compared with earlier works irrespective of the different roughness at top and bottom face.

Fig. 5 shows a comparison of obtained capacity factors ($N_u = V_u/s_u B$) for the lowest value of v/k considered ($= 0.01$) with that obtained under NT conditions from earlier studies using numerical modelling (Yu et al. [14]), limit analysis (Merifield et al. [5]) and model testing (Das [32]; Singh and Ramaswamy [11]). Similarly, the results obtained for the highest pull-out rate ($v/k = 1000$) are compared with results available in literature for FT conditions (Yu et al. [14], Chen et al. [13]). Merifield et al. [5] presented lower and upper bound limit analysis results that neglected tension (i.e. “No Tension” (NT) conditions), and Chen et al. [13] reported a limiting value of uplift factor ($N_{u(\text{limit})}$) close to 11.8 from centrifuge model tests. Additional LDFE analyses were also carried out for NT and FT conditions using the approach adopted by Maitra et al. [18] which does not include the ‘gap’ elements and these results are depicted in Fig. 5.

As can be seen from the figure, the results for $v/k = 0.01$ are in good agreement with the NT factors whereas, N_u obtained for $v/k = 1000$ are in accord with FT factors obtained from earlier studies. These results provide a validation of the adopted modelling approach.

4. Results and discussions

4.1 Soils with constant s_u

Fig. 6(a) shows an example of variation of uplift resistance (V) with anchor displacement (δ) for different pull-out rates ($w = 2$ m, $B = 1$ m, $s_u = 10$ kPa, $\gamma' = 0$). The results are here also compared with the “No Tension” and “Full Tension” capacities obtained separately through additional LDFE analyses that did not include the gap elements and seepage flow. From the figure it can be seen that there is an increase in resistance with increase in v/k which is due to

the generation of negative excess pore pressure below the anchor at higher pull-out rates. The capacities at very low uplift rate (say $v/k = 0.01 - 0.1$) are in close agreement with NT capacities, whereas, FT capacity matches well with high pull-out rate cases (say, $v/k \geq 50$). Similar resistance-displacement plots are obtained for various cases considered (a particular case is shown in Fig. 6(a)) and uplift capacity factors (N_u) are calculated from peak uplift resistances (V_u) for these cases.

A comparison of N_u for various values of $\gamma'B/s_u$ (0, 1 and 2) and $B = 1$ m is shown in Fig. 6(b) (for $w/B = 2$) and Fig. 6(c) (for $w/B = 4$). From the figures it can be seen that there is an increase in NT capacities (i.e., $v/k \approx 0$) with increase in $\gamma'B/s_u$. This happens because breakaway occurs for these cases and soil above the anchor is lifted upwards. Thus, larger force is required for heavier soil. On the other hand, under FT conditions (here, high v/k), soil buoyancy – which contributes an uplift force of $V_{\text{buoy}} = \gamma'tB$ – assists uplift and hence, there is a reduction in capacity with increase in $\gamma'B/s_u$. However, because of the smaller cross-sectional area of the thin anchors considered here ($t/B = 0.1$), this decrease is negligible.

The results also suggest that with an increase in $\gamma'B/s_u$, there is a decrease in the difference between FT and NT capacities for anchors placed at a particular embedment. For $(w/B) \geq (w/B)_{\text{limit}}$ (see $\gamma'B/s_u = 2$ in Fig. 6(c)), this difference reduces to zero and uplift capacity becomes independent of v/k . This is because the self-weight of the soil is sufficiently high relative to the strength to prevent any gap beneath the anchor plate remaining stable. The effects of buoyancy on uplift capacity have been studied in detail by earlier researchers (Martin and White [17]; Maitra et al. [18]; Singh et al. [15]).

Fig. 7 shows a plot of equipotential lines and flow directions for $v/k = 5$ and 50 at $w = 2$ m and $B = 1$ m. Here, the total head is calculated assuming mudline as the datum. From the figure, it can be observed that higher negative excess pore pressure is developed within the

soil at higher pull-out rates (compare total heads for $v/k = 5$ and 50). The flow directions (the arrows in Fig. 7 shows only the direction and not magnitude of fluid velocity) at various nodes of the FE mesh suggests that water enters through the seabed into the soil and the flow occurs towards the gap below the anchor. Also, it becomes evident from the figure that the gap stretches more for $v/k = 5$ as compared to $v/k = 50$ which suggests that breakaway occurs for the slower pull-out rate.

Some typical examples of uplift mechanisms are presented in Fig. 8 for various pull-out rates, normalised buoyant unit weight of soil and embedment ratio. Buoyancy and interface tension governs the feasibility of breakaway. With an increase in pull-out rate, greater interface tension is mobilised which can result in a reverse bearing mechanism involving soil lying both below and above the anchor (compare mechanisms for $v/k = 10$ and 100 in Fig. 8(a)). On the other hand, the same reverse bearing mechanism can occur even at lower pull-out rates for soils with greater self-weight (see Fig. 8(b)) because of the larger overburden stress that tends to drive collapse of soil into any gap beneath the anchor.

At shallow embedment, the failure mechanisms extend up to the mudline and are termed global shear failure. However, for anchors placed at greater depth, the mechanisms become fully localised when gap formation is not feasible at high pull-out rates (see Fig. 8(c) for $v/k = 100$). Under such circumstances, a local flow of soil occurs predominantly from top of anchor towards anchor base.

Fig. 9 shows some examples of distribution of negative excess pore pressures for various embedment, pullout rates and normalised submerged unit weight. It should be noted that the pore pressure distributions shown in the figure does not consider the effect of change in void ratio of soil during flow and considers steady state seepage due to head difference only. The figure substantiates the fact that higher interface tension is generated for higher v/k as long as

FT conditions are not reached. Fig. 9(d) shows a case of $(w/B) > (w/B)_{\text{limit}}$ in which the uplift mechanisms are same at all pullout rates (see Fig 8(d)). In this case, breakaway does not occur even for slower pullout and as a result NT and FT capacities are equal. Thus, interface tension is not generated as uplift capacity has reached its limiting value even under NT conditions.

By realistically modelling seepage flow into a water-filled gap, it is possible to also identify an intermediate mechanism comprising of a combination of the global shear mechanism (involving uplift of soil column above anchor) and local shear flow at intermediate pull-out rates (see mechanism for $v/k = 10$ in Fig. 8(a)). The local flow of soil around the anchor edges takes place due to seepage force which drags the soil lying near the anchor edges from top to bottom. This occurs when the seepage force is not adequate to generate sufficient interface tension required for reverse end bearing along the entire anchor width, and instead the seepage flow allows water to fill this opening gap. As mentioned earlier, for $(w/B) \geq (w/B)_{\text{limit}}$ (say $w/B = 2$, $\gamma'B/s_u = 2$) the capacity becomes independent of v/k as buoyancy alone prevents breakaway resulting in identical failure mechanisms at all pull-out rates (see Fig. 8(d)).

4.2 Determination of Hvorslev's intake factors

The transition from NT to FT conditions via a progressive increase in seepage force mobilised beneath the anchor can be analysed by modelling the flow of water into the gap beneath the anchor. The situation is analogous to the constant head borehole permeability test with the anchor width corresponding to the borehole width. The anchor velocity multiplied by its width is equivalent to the rate of volumetric flow into the borehole. The interface tension mobilised beneath the anchor, T , can be solved using Hvorslev's general equation (Hvorslev [33]). If p is the magnitude of negative excess pore pressure acting at the anchor base, then the interface tension, T (i.e., the seepage force) can be written as:

$$(1) \quad T = pB = \gamma_w h_p B$$

where, h_p is the drop in pressure head and γ_w is the unit weight of water. For a given flow rate or anchor velocity, h_p depends on a geometric constant known as a shape or (Hvorslev's) intake factor (F). h_p can be expressed as:

$$(2) \quad h_p = \frac{Q}{kF} = \frac{vB}{kF}$$

where, Q is the steady state volumetric flow rate, which is equivalent (in plane strain) to vB .

Using Eqs. (1) and (2), the interface tension can be finally expressed as:

$$(3) \quad T = \left(\frac{\gamma_w B^2}{F} \right) \left(\frac{v}{k} \right)$$

F depends principally on (a) the geometry of the flow domain and the cavity, (b) the boundary conditions, and (c) soil permeability anisotropy. In the present study, soil permeability has been considered to be isotropic. Several studies exist on determination of F for various geometries, primarily related to an axisymmetric borehole (Hvorslev [33]; Wilkinson [34]; Lowther, [35]; Brand and Premchitt [36], [37]; Ratnam et al. [38]). From these studies, it becomes evident that rate of dissipation of excess pore pressure varies with problem geometry and this rate will be more for pullout of plate anchors with lower values of aspect ratios as compared to strip anchors. Thus, higher pullout velocities may be required for mobilising same amount of interface tension on rectangular plate anchors as compared to strip anchors.

In the present study, which considers plane strain conditions, F is found empirically by curve-fitting for various values of w and B (see Figs. 10(a) and (b)). From the figures it can be seen that F is dependent only on the ratio w/B (F is same for $B = 0.5$ m and 1 m at $w/B = 4$), following the relationship shown in Fig. 10(c). The shape factor decreases with w/B until it reaches a near-constant value at deep embedment. This is expected because at greater

embedment, the flow path is longer so requires a greater head for a given flow rate or anchor velocity. However, a limiting condition is reached at depth when the flow field is independent of the distant ground surface. The shape factor can be expressed as:

$$(4) \quad F = \frac{1.5}{1 - \exp\left[-1.1(w/B)^{0.65}\right]}$$

Alternative relationships for the seepage force were considered, allowing the width of the equivalent borehole to vary as a fraction of the anchor width. In this case, the soil resistance was calculated as a combination of flow around the edge of the anchors and shearing of a narrower chimney of soil above the plate. However, adding this further variable to the failure mechanism did not lead to a more optimal (lower) calculated capacity.

Having established a relationship for the seepage force for different rates of anchor movement and thus gap opening, it is possible to combine this with the conventional NT and FT bearing capacity solutions to determine a general expression for anchor uplift capacity considering seepage-induced gap formation.

4.3 Effects of strain rate-dependent soil strength on uplift capacity

Since the present study focuses on studying uplift capacity at various pullout rates, it becomes important to consider the effect of strain rate on the undrained shear strength of the soil. For this purpose, the undrained shear strength at the Gauss points is modified after each small incremental displacement of anchor based on the maximum shear strain rate using the approach proposed by Einav and Randolph [26] and Zhou and Randolph [27]. The undrained shear strength of soil is expressed using the rate dependent soil model given by:

$$(5) \quad s_u = s_{u0} \left[1 + \mu \log \left(\frac{\max(|\dot{\gamma}_{\max}|, \dot{\gamma}_{\text{ref}})}{\dot{\gamma}_{\text{ref}}} \right) \right]$$

Here, μ is rate of strength increase per decade; with suggested range of 0.05–0.2 (Dayal and Allen [39]; Graham et al. [40]; Biscontin and Pestana [41]). $\dot{\gamma}_{\text{ref}}$ is the reference shear strain rate, 3×10^{-6} ($\sim 1\%$ per hour) and s_{u0} is the shear strength measured at the reference shear strain rate. The maximum rate of shear strain expressed as:

$$(6) \quad \dot{\gamma}_{\text{max}} = \frac{\Delta \varepsilon_1 - \Delta \varepsilon_3}{\delta / B} \frac{v}{B}$$

Here, $\Delta \varepsilon_1$, $\Delta \varepsilon_3$ are major and minor principal strains, v is anchor velocity and δ is small incremental displacement.

In the present study, μ was varied from 0 to 0.15 at interval of 0.05. The obtained normalised uplift capacities (normalised with respect to $s_{u0}B$) corresponding to various values of μ for $w/B = 2$ and 4 are shown in Figs. 11 (a) and (b) respectively. From the figure it can be seen that uplift capacities increase significantly at higher pull-out rates which is attributed to the viscous effects at higher pullout velocities. Several researchers (Chatterjee et al. [42]; Ghorai and Chatterjee [43]) used the approach of normalising capacity factors with $s_{u,\text{eq}}B$ where $s_{u,\text{eq}}$ is the equivalent shear strength which considers the increase in shear strength with increase in strain rate. In these studies, $s_{u,\text{eq}}$ has been expressed as:

$$(6) \quad s_{u,\text{eq}} = s_{u0} \left[1 + \mu \log \left\{ \max \left(1, \frac{f_r v}{B \dot{\gamma}_{\text{ref}}} \right) \right\} \right]$$

Here, f_r represents the average operative shear strain rate acting during the uplift process. Using a similar approach, on assuming a value of f_r as 0.5 for all embeddings, the various curves obtained for various values of μ (see Figs. 11(a) and (b)) reduce to a single curve for a particular embedment (see Fig. 11(c)). Equation 6 therefore provides a simple basis for

allowing for the effect of strain rate on soil strength in the solutions for uplift resistance described in the next section.

4.4 Prediction equations for obtaining capacity under FT and NT conditions

Singh et al. [15] proposed FT and NT capacity prediction model for anchors placed at various inclinations. From their study, the FT and NT capacities (V_{FT} and V_{NT} respectively) for horizontal anchors can be calculated using Eqs. (7) and (8).

$$(7) \quad \begin{aligned} V_{FT} &= V_{\text{flow}} - V_{\text{buoy}} \\ &= s_{u,FT} B \left[5.14 + (11.78 - 5.14) \left\{ 1 - \exp \left(-0.8 \left(w / B \right)^{1.6} \right) \right\} \right] - \gamma' B t \end{aligned}$$

$$(8) \quad \begin{aligned} V_{NT} &= V_{\text{slip}} + V_{\text{cover}} \\ &= s_{u,NT} B \left[11.78 \left\{ 1 - \frac{1}{1 + 0.22 \left(w / B \right)} \right\} \right] + \gamma' B H \leq V_{FT} \end{aligned}$$

Here, V_{flow} is the component of uplift resistance mobilised from shearing in a flow-round mechanism whereas, V_{slip} is the component of uplift resistance mobilised from shear resistance acting along failure planes extending above the anchor for the cases of breakaway. V_{cover} is the resistance from the soil effective weight above the anchor. Under FT conditions, V_{buoy} is the buoyancy of the soil displaced by the anchor itself, $\gamma' t B$. $(s_{u,eq})_{FT}$ and $(s_{u,eq})_{NT}$ are the average equivalent shear strength of soil involved in the uplift mechanism under FT and NT conditions respectively and these can be obtained using empirical factors f_{NBA} and f_{BA} (proposed by Singh et al. [15]) in Eqs. (9) and (10). While calculating these equivalent shear strengths $((s_{u,eq})_{FT}$ and $(s_{u,eq})_{NT})$, effect of strain rate is also incorporated here using Eq. (6).

$$(9) \quad (s_{u,eq})_{FT} = \left\{ s_{um} + f_{NBA} s_{ug} z \right\} \cdot \left[1 + \mu \log \left\{ \max \left(1, \frac{0.5v}{B \dot{\gamma}_{\text{ref}}} \right) \right\} \right]$$

$$(10) \quad (s_{u,eq})_{NT} = \{s_{um} + f_{BA}s_{ug}z\} \cdot \left[1 + \mu \log \left\{ \max \left(1, \frac{0.5v}{B\dot{\gamma}_{ref}} \right) \right\} \right]$$

Here, s_{um} is the shear strength at mudline, whereas, s_{ug} is the shear strength gradient along depth. As there is a transition from global shear mechanism to local flow mechanism with increase in embedment, values of f_{NBA} and f_{BA} approach 1 with increase in embedment because in case of a local flow mechanism the average s_u within the failure zone is equal to s_u at the anchor base level (because the mechanism extends equally from the top and bottom face of anchor). For cases where failure zone extends to the mudline, this average s_u is generally less than s_u at anchor level because the volume of soil involved during uplift is above the anchor, and has a lower strength, so f_{NBA} and f_{BA} generally have values less than 1. These aspects are discussed in detail by Singh et al. [15] and the factors f_{NBA} and f_{BA} can be obtained using charts proposed from their study.

The uplift capacity at various pull-out rates is the minimum of the FT capacity and a summation of NT capacities and seepage-induced interface tension (T) (see Eq. 11) where, T can be computed using Eqs. (3) and (4). The entire prediction methodology is summarised in Fig. 12.

$$(11) \quad V_u = V_{NT} + T \leq V_{FT}$$

4.5 Generalisation of the prediction equations for various shear strength profiles

In order to check the accuracy of the prediction equations for various site conditions, it has been checked against ranges of realistic soil properties encountered in the field. For this purpose, several combinations of various parameters were chosen (see Table 1) and the capacities determined using the prediction model ($V_{predicted}$) are compared with the numerical

1 results (V_{obtained}) in Fig. 13. The anchor width was assumed to be 1 m and γ' was taken as 6
2 kN/m^3 in these set of analyses. Embedment ratio, v/k and rate parameter μ were varied over
3
4 wide ranges. Two different types of shear strength profile were also considered which
5
6 correspond to soil heterogeneity factors, $\kappa (= s_{\text{ug}}B/s_{\text{um}})$ of 0 and 10. Fig. 13 shows that the
7
8 prediction model works well for various soil properties, pull-out rates and embedment ratios.
9
10 The magnitude of average and maximum absolute error in prediction of capacity for these
11
12 cases is found to be 3.0 % and 8.2 % respectively.
13
14
15
16

17
18 This calculation method can be used to determine the resistance when an anchor is pulled
19
20 from the seabed at rates that lie between the NT and FT cases. Equally, it can be used to
21
22 determine the rate at which an anchor will fail – via a seepage-induced gap opening behind it
23
24 – when subjected to a sustained load ($V_{\text{sustained}}$) that exceeds the NT limit but lies below the
25
26 FT case.
27
28

29
30 Let us consider an example anchor of width 2 m buried at $w/B = 4$ subjected to varying
31
32 magnitudes of $V_{\text{sustained}}$. $V_{\text{sustained}}$ can be expressed as a fraction of initial available seepage
33
34 force and is denoted here using a factor $\eta = (V_{\text{sustained}} - V_{\text{NT}})/(V_{\text{FT}} - V_{\text{NT}})$. Four cases of η
35
36 ranging from 0.1 to 0.8 have been considered in soil with a strength profile of $s_u = 1.25z$ kPa
37
38 (z is depth measured from mudline), $\gamma' = 4 \text{ kN/m}^3$ and permeability of 10^{-9} m/s . At these load
39
40 levels, the anchor starts displacing at a velocity lying in the range of 0.034 – 0.27 mm/day
41
42 (see Fig. 14(a)) as seepage flow fills the void behind it. If the anchor continues to uplift at this
43
44 initial rate, it would take the anchor years to reach the mudline. However, in reality the rate of
45
46 pull-out progressively increases as the anchor moves upwards into softer soil. The solution in
47
48 this paper can be applied in an incremental form to track the growth of a seepage-induced gap
49
50 behind an anchor, and therefore model this acceleration and destabilisation of the anchor as it
51
52 is pulled upwards into weaker soil, which in turns causes additional load to be carried via
53
54
55
56
57
58
59
60
61
62
63
64
65

negative excess pore pressure beneath the anchor, which in turn raises the seepage rate. The change in anchor velocity and embedment with time has been obtained using the proposed solution in incremental form and is shown in Fig. 14. The increase in velocity as anchor enters into the softer soil during pull-out is captured well in the figure. These trends mirror the experimental observations reported by Han et al. [19].

5. Conclusions

In the present work, the effect of seepage and gap formation below a plate anchor subjected to uplift has been studied. In order to numerically simulate a gap filled with water, a thin layer of soft porous elements has been placed below the anchor base. The necessity of resetting stresses within the gap elements to zero after every small incremental displacement of LDPE analysis has been highlighted in the paper and thus, the present methodology is able to simulate the phenomenon of gap formation more realistically as compared to earlier studies. From the study, the component of anchor uplift capacity related to interface tension (T) is found to depend on pull-out velocity (v), soil permeability (k) and embedment ratio (w/B). By analogy to the constant head borehole seepage test, T has been expressed in terms of Hvorslev's intake factor (F). The uplift capacity at various pull-out rates is then expressed by superimposing the geotechnical shear resistance, buoyancy force and seepage force. The present study also considers effect of strain rate on s_u and its effect on uplift capacity.

It has been seen that for embedment ratio greater than a limiting embedment ratio ($w/B \geq (w/B)_{\text{limit}}$), uplift capacity becomes independent of pull-out rate. A design flowchart has been proposed from the results obtained from the current work. The accuracy of the prediction model has been checked by exploring several combinations of influential parameters that are often encountered in field. The model works well with the average discrepancy compared to the numerical results being less than 3 %. Thus, the results obtained from the study are useful

for design practice as they quantify the effect of interface tension on uplift capacity. The calculation method can be used to determine the longevity of an anchor subjected to sustained load that exceeds the no-tension capacity. It can also be applied incrementally to capture the progressive acceleration and destabilisation of an anchor that is progressively pulled upwards into softer soil as a water-filled gap grows behind it. Thus, the present study can help a designer in estimating the displacement that can accumulate over the period for which a design load is applied.

Acknowledgements

The authors would like to acknowledge the financial support received through grant number AIC122-2015 from Australia-India Council, Department of Foreign Affairs and Trade, Australian Government for carrying out this collaborative research between the University of Western Australia and Indian Institute of Technology Bombay, India.

List of symbols

α	Interface roughness co-efficient
B	Width of anchor
E_{soil}	Deformation modulus of soil
E_g	Deformation modulus of gap
η	Factor to express sustained load in excess of V_{NT} as a fraction of initial available seepage force
$\varepsilon_1, \varepsilon_3$	Major and minor principal strains respectively
F	Hvorslev's intake or shape factor
$f_{\text{NBA}}, f_{\text{BA}}$	Empirical factor for obtaining $s_{\text{u,FT}}$ and $s_{\text{u,NT}}$ respectively
f_t	Average operative shear strain rate
γ_w	Unit weight of water
γ'	Buoyant unit weight of soil
$\dot{\gamma}_{\text{ref}}$	Reference shear strain rate
$\dot{\gamma}_{\text{max}}$	Maximum rate of shear strain
H	Soil cover height
h_p	Pressure head acting at anchor base
k	Permeability of soil
k_g	Permeability of gap
κ	Soil heterogeneity factor (kD/s_{um})
N_u	Uplift capacity factor
$N_{\text{u(limit)}}$	Limiting value of N_u
$N_{\text{FT}}, N_{\text{NT}}$	Uplift capacity factor under FT and NT conditions respectively
p	Suction pressure acting at anchor base
Q	Steady state flow rate into the gap below the anchor
s_u	Undrained shear strength of soil
s_{ug}	Shear strength gradient along depth
s_{um}	s_u at seabed mudline
$s_{\text{u,eq}}$	Equivalent undrained shear strength considering strain rate effect
$(s_{\text{u,eq}})_{\text{FT}}, (s_{\text{u,eq}})_{\text{NT}}$	Average s_u in the failure zone under FT and NT conditions respectively
s_{u0}	Shear strength measured at the reference shear strain rate

T	Interface tension
t	Thickness of anchor
t_g	Initial thickness of the gap
u_{anchor}	Displacement of anchor
u_{soil}	Displacement of soil
μ	Rate of strength increase per decade
V	Uplift resistance
$V_{\text{sustained}}$	Sustained load applied to an anchor
V_{buoy}	Contribution of soil weight towards uplift capacity under FT conditions
V_{cover}	Uplift resistance from effective weight of soil cover under breakaway conditions
V_{flow}	Shear resistance mobilised on failure planes for flow-round mechanism
$V_{\text{FT}}, V_{\text{NT}}$	Uplift capacity under full tension and no tension conditions respectively
V_{slip}	Shear resistance acting on slip planes for breakaway mechanism
V_u	Uplift capacity i.e., peak uplift resistance
v	Pull-out velocity of anchor
w	Embedment depth of anchor base
w_{limit}	w at which N_u reaches its FT capacity under NT conditions
z	Depth measured from mudline

List of abbreviations

FT	Full tension
LDFE	Large deformation finite element
NT	No tension
RITSS	Remeshing and interpolation technique with small strain
SSFE	Small strain finite element
SPR	Superconvergent patch recovery

References

- [1] Randolph M, Gourvenec, S. Offshore geotechnical engineering. CRC Press; 2011.
- [2] Vesic AS. Breakout resistance of objects embedded in ocean bottom. *Journal of Soil Mechanics and Foundation Engineering*, ASCE 1971; 97(9):1183-203.
- [3] Rowe RK. Soil structure interaction analysis and its application to the prediction of anchor behaviour. Ph.D. thesis: University of Sydney, Sydney, Australia; 1978.
- [4] Rao KSS, Kumar J. Vertical uplift capacity of horizontal anchors. *Journal of Geotechnical Engineering* 1994;120(7):1134-46. [https://doi.org/10.1061/\(ASCE\)0733-9410\(1994\)120:7\(1134\)](https://doi.org/10.1061/(ASCE)0733-9410(1994)120:7(1134)).
- [5] Merifield RS, Sloan SW, Yu HS. Stability of plate anchors in undrained clay. *Géotechnique* 2001;51(2):141–53. <https://doi.org/10.1680/geot.2001.51.2.141>.
- [6] Merifield RS, Lyamin AV, Sloan SW, Yu HS. Three-dimensional lower bound solutions for stability of plate anchors in clay. *Journal of Geotechnical and Geoenvironmental Engineering* 2003;129(3):243–53. [https://doi.org/10.1061/\(ASCE\)1090-0241\(2003\)129:3\(243\)](https://doi.org/10.1061/(ASCE)1090-0241(2003)129:3(243)).
- [7] Merifield RS, Lyamin AV, Sloan SW. Stability of inclined strip anchors in purely cohesive soil. *Journal of Geotechnical and Geoenvironmental Engineering* 2005;131(6):792–9. [https://doi.org/10.1061/\(ASCE\)1090-0241\(2005\)131:6\(792\)](https://doi.org/10.1061/(ASCE)1090-0241(2005)131:6(792)).
- [8] Choudhury D, Rao KSS. Seismic uplift capacity of inclined strip anchors. *Canadian Geotechnical Journal* 2005;42(1):263-71. <https://doi.org/10.1139/t04-074>.
- [9] White DJ, Cheuk CY, Bolton MD. The uplift resistance of pipes and plate anchors buried in sand. *Géotechnique* 2008;58(10):771-9. <https://doi.org/10.1680/geot.2008.3692>.
- [10] Rangari SM, Choudhury D, Dewaikar DM. Seismic uplift capacity of shallow horizontal strip anchor under oblique load using pseudo-dynamic approach. *Soils and Foundations* 2013;53(5):692-707. doi: 10.1016/j.sandf.2013.08.007.

- [11] Singh SP, Ramaswamy SV. Effect of shape on holding capacity of plate anchors buried in soft soil. *Geomechanics and Geoengineering* 2008;3(2):145–54. <https://doi.org/10.1080/17486020802126875>.
- [12] Wang D, O’Loughlin CD. Numerical study of pull-out capacities of dynamically embedded plate anchors. *Canadian Geotechnical Journal* 2014;51(11):1263–72. <https://doi.org/10.1139/cgj-2013-0485>.
- [13] Chen J, Leung CF, Chen Z, Tho KK, Chow YK. Centrifuge model study on pull-out behaviour of plate anchors in clay with linearly increasing strength. In: *Frontiers in offshore geotechnics III: ISFOG 2015*, Taylor & Francis Group, London, 2015, p. 839–44.
- [14] Yu L, Liu J, Kong XJ, Hu Y. Numerical study on plate anchor stability in clay. *Géotechnique* 2011;61(3):235–46. <https://doi.org/10.1680/geot.8.P.071>.
- [15] Singh V, Maitra S, Chatterjee S. Generalized Design Approach for Inclined Strip Anchors in Clay. *International Journal of Geomechanics, ASCE* 2017;17(6):04016148. [https://doi.org/10.1061/\(ASCE\)GM.1943-5622.0000849](https://doi.org/10.1061/(ASCE)GM.1943-5622.0000849).
- [16] Liu J, Tan M, Hu Y. New analytical formulas to estimate the pullout capacity factor for rectangular plate anchors in NC clay. *Applied Ocean Research* 2018;75:234–47. <https://doi.org/10.1016/j.apor.2018.04.002>.
- [17] Martin CM, White DJ. Limit analysis of the undrained bearing capacity of offshore pipelines. *Géotechnique* 2012;62(9):847–63. <https://doi.org/10.1680/geot.12.OG.016>.
- [18] Maitra S, Chatterjee S, Choudhury, D. Generalized framework to predict undrained uplift capacity of buried offshore pipelines. *Canadian Geotechnical Journal* 2016;53(11):1841–52. <https://doi.org/10.1139/cgj-2016-0153>.
- [19] Han C, Wang D, Gaudin C, O’Loughlin CD, Cassidy MJ. Behaviour of vertically loaded plate anchors under sustained uplift. *Géotechnique* 2016;66(8):681–93. <https://doi.org/10.1680/jgeot.15.P.232>.

- [20] Cao J. Centrifuge modelling and numerical analysis of the behaviour of suction caissons in clay. Ph.D. thesis: Memorial University of Newfoundland, St. John's, Newfoundland, Canada; 2003.
- [21] Mana DS, Gourvenec S, Randolph MF. Numerical modelling of seepage beneath skirted foundations subjected to vertical uplift. *Computers and Geotechnics* 2014;55:150-7. <https://doi.org/10.1016/j.compgeo.2013.08.007>.
- [22] DNV. Design and installation of plate anchors in clay. Recommended Practice E302, Det Norske Veritas (DNV), Norway; 2017.
- [23] Carter JP, Randolph MF, Wroth CP. Stress and pore pressure changes in clay during and after the expansion of a cylindrical cavity. *International Journal for Numerical and Analytical Methods in Geomechanics* 1979;3(4):305-22. <https://doi.org/10.1002/nag.1610030402>.
- [24] Gourvenec SM, Vulpe C, Murthy TG. A method for predicting the consolidated undrained bearing capacity of shallow foundations. *Géotechnique* 2014;64(3):215-25. <https://doi.org/10.1680/geot.13.P.101>.
- [25] Zhou Z, White DJ, O'Loughlin CD. An effective stress framework for estimating penetration resistance accounting for changes in soil strength from maintained load, remoulding and reconsolidation. *Geotechnique* 2018: published ahead of print 16 March 2018. doi: <http://dx.doi.org/10.1680/jgeot.17.P.217>.
- [26] Einav I, Randolph MF. Combining upper bound and strain path methods for evaluating penetration resistance. *Int J Numer Meth Eng* 2005;63(14):1991–2016. <http://dx.doi.org/10.1002/nme.1350>.
- [27] Zhou H, Randolph MF. Computational techniques and shear band development for cylindrical and spherical penetrometers in strain-softening clay. *International Journal of Geomechanics* 2007;7(4): 287-95. [https://doi.org/10.1061/\(ASCE\)1532-3641\(2007\)7:4\(287\)](https://doi.org/10.1061/(ASCE)1532-3641(2007)7:4(287)).

- [28] Hu Y, Randolph MF. A practical numerical approach for large deformation problems in soil. International Journal for Numerical and Analytical Methods in Geomechanics 1998;22(5):327–50. [https://doi.org/10.1002/\(SICI\)1096-9853\(199805\)22:5%3C327::AID-NAG920%3E3.0.CO;2-X](https://doi.org/10.1002/(SICI)1096-9853(199805)22:5%3C327::AID-NAG920%3E3.0.CO;2-X).
- [29] Hu Y, Randolph MF. H-adaptive FE analysis of elasto-plastic nonhomogeneous soil with large deformation. Computers and Geotechnics 1998;23(1–2):61–83. [https://doi.org/10.1016/S0266-352X\(98\)00012-3](https://doi.org/10.1016/S0266-352X(98)00012-3).
- [30] Dassault Systèmes. ABAQUS 6.13 analysis user's guide. Providence, RI, USA: Simulia Corp.; 2013.
- [31] Zienkiewicz OC, Zhu JZ. The superconvergent patch recovery and a posteriori error estimates. Part 1: The recovery technique. International Journal for Numerical Methods in Engineering 1993;33:1331–64. <https://doi.org/10.1002/nme.1620330702>.
- [32] Das BM. Model tests for uplift capacity of foundations in clay. Soils and Foundations 1978;18(2):17–24. https://doi.org/10.3208/sandf1972.18.2_17.
- [33] Hvorslev MJ. Time lag and soil permeability in ground-water observations. US Army Corps of Engineers, Waterways Experiment Station Bulletin No. 36, Mississippi, USA; 1951.
- [34] Wilkinson WB. Constant head in situ permeability tests in clay strata. Géotechnique 1968;18(2):172–94. <https://doi.org/10.1680/geot.1968.18.2.172>.
- [35] Lowther G. A note on Hvorslev's intake factors. Géotechnique 1978;28(4):465–6. <https://doi.org/10.1680/geot.1978.28.4.465>.
- [36] Brand EW, Premchitt J. Shape factors of cylindrical piezometers. Géotechnique 1980;30(4):369–84. <https://doi.org/10.1680/geot.1980.30.4.369>.
- [37] Brand EW, Premchitt J. Shape factors of some non-cylindrical piezometers. Géotechnique 1980;30(4):536–7. <https://doi.org/10.1680/geot.1980.30.4.536>.

- [38] Ratnam S, Soga K, Whittle, RW. Revisiting Hvorslev's intake factors using the finite element method. *Géotechnique* 2001;51(7):641-5. <https://doi.org/10.1680/geot.2001.51.7.641>.
- [39] Dayal U, Allen JH. The effect of penetration rate on the strength of remolded clay and sand samples. *Canadian Geotechnical Journal* 1975;12(3):336–348. <https://doi.org/10.1139/t75-038>.
- [40] Graham J, Crooks JHA, Bell AL. Time effects on the stress-strain behaviour of natural soft clays. *Géotechnique* 1983;33(3):327–340. <https://doi.org/10.1680/geot.1983.33.3.327>.
- [41] Biscontin G, Pestana JM. Influence of peripheral velocity on vane shear strength of an artificial clay. *Geotechnical Testing Journal* 2001;24(4):423–9. <https://doi.org/10.1520/GTJ11140J>.
- [42] Chatterjee S, Randolph MF, White DJ. The effects of penetration rate and strain softening on the vertical penetration resistance of seabed pipelines. *Géotechnique* 2012;62(7):573–82. <http://dx.doi.org/10.1680/geot.10.P.075>.
- [43] Ghorai B, Chatterjee S. Influences of strain rate and soil remoulding on initial break-out resistance of deepwater on-bottom pipelines. *Computers and Geotechnics* 2017;91:82-92. <https://doi.org/10.1016/j.compgeo.2017.07.006>.

List of Tables

Table 1 List of input parameters for testing robustness of the prediction model

Table 1 List of input parameters for testing accuracy of the prediction model

Anchor embedment ratio, w/B	Rate of strength increase per decade, μ	Shear strength at mudline, s_{um} (kPa)	Shear strength gradient along depth, s_{ug} (kPa/m)	Soil heterogeneity factor, $\kappa =$ $s_{ug}B/s_{um}$	Normalised pull-out rate, v/k
2	0.05, 0.10, 0.15	20	0	0	0.1, 1, 10, 100, 1000
		0.5	5	10	
4	0.05, 0.10, 0.15	20	0	0	0.1, 1, 10, 100, 1000
		0.5	5	10	
7	0.05, 0.10, 0.15	20	0	0	0.1, 1, 10, 100, 1000
		0.5	5	10	

Illustration Captions

Fig. 1 Simplified representation of the problem geometry

Fig. 2 A typical FE mesh details for $w/B = 2$ showing the anchor, soil and gap parts of the numerical model

Fig. 3 Comparison of uplift mechanisms obtained with and without gap elements at $v/k = 0.01$ and 1000 ($w/B = 4$, $\gamma'B/s_u = 0$)

Fig. 4 Normalised uplift resistance ($V/s_u B$) with and without resetting of stresses to zero for $E_g = 1$ and 10 kPa, $v/k = 0.01$ and 1000

Fig. 5 Comparison of N_u obtained for $v/k = 0.01$ and 1000 in weightless soils with NT and FT factors available in literature

Fig. 6 Typical examples of obtained uplift response for $B = 1$ m: (a) Uplift resistance – displacement plot for $w/B = 2$, $\gamma'B/s_u = 0$; (b) N_u versus v/k for $w/B = 2$, $\gamma'B/s_u = 0, 1, 2$ (c) N_u versus v/k for $w/B = 4$, $\gamma'B/s_u = 0, 1, 2$

Fig. 7 Equipotential lines and flow directions for $v/k = 5$ and 50 at $w/B = 2$, $B = 1$ m

Fig. 8 Displacement contour diagrams for $w/B = 2, 4$; $\gamma'B/s_u = 0, 2$; $v/k = 10, 100$; $B = 1$ m

Fig. 9 Negative excess pore pressure contour diagrams for $w/B = 2, 4$; $\gamma'B/s_u = 0, 2$; $v/k = 10, 100$; $B = 1$ m

Fig. 10 Determination of shape factor, F for predicting N_u at various pull-out rates: (a) $w/B = 4$, $B = 0.5$ m; (b) $w/B = 4$, $B = 1$ m; and (c) Obtained values of F versus w/B

Fig. 11 Normalised uplift capacities for $\mu = 0, 0.05, 0.10$ and 0.15 ($s_u = 10$ kPa, $B = 1$ m, $\gamma' = 0$, $\dot{\gamma}_{\text{ref}} = 3 \times 10^{-6} \text{ s}^{-1}$, $k = 10^{-7} \text{ m/s}$): (a) V_u normalised with $s_{u0}B$ for $w/B = 2$; (b) V_u normalised with $s_{u0}B$ for $w/B = 4$; and (c) V_u normalised with $s_{u,eq}B$ for $w/B = 2, 4$

Fig. 12 Methodology for predicting uplift capacity of strip anchor allowing for seepage-induced gap formation

Fig. 13 Parametric testing of analytical solution to confirm accuracy across parameter space

Fig. 14 Prediction of pull-out velocity and embedment for anchors subjected to sustained loads using proposed solution in incremental form ($B = 2$ m, $w_{\text{initial}} = 8$ m, $k = 10^{-9}$ m/s, $s_u = 1.25z$ kPa, $\gamma' = 4$ kN/m³)

Figure1

Figure2

[Click here to download high resolution image](#)

Figure3
[Click here to download high resolution image](#)

Figure 5

- NT, Yu et al. (2011) — FT, Yu et al. (2011)
- - - NT - Lower Bound, Merifield et al. (2001) NT - Upper Bound, Merifield et al. (2001)
- × Das (1978) + Singh and Ramaswamy (2008)
- NT – LDFE, Present Study ▲ FT – LDFE, Present Study
- $v/k = 0.01$ – LDFE, Present Study Δ $v/k = 1000$ – LDFE, Present Study

Figure 6

(a)

(b)

(c)

Figure7

[Click here to download high resolution image](#)

Figure8

[Click here to download high resolution image](#)

Figure9

[Click here to download high resolution image](#)

Figure 10

(a)

(b)

(c)

(a)

(b)

(c)

Figure12

Figure 13 $w/B = 2$ $w/B = 4$ $w/B = 7$

Line of parity

$\pm 5\%$ error $\pm 10\%$ error

Figure 14

(a)

(b)

Minerva Access is the Institutional Repository of The University of Melbourne

Author/s:

Maitra, S;White, D;Chatterjee, S;Choudhury, D

Title:

Numerical modelling of seepage and tension beneath plate anchors

Date:

2019-04-01

Citation:

Maitra, S., White, D., Chatterjee, S. & Choudhury, D. (2019). Numerical modelling of seepage and tension beneath plate anchors. COMPUTERS AND GEOTECHNICS, 108, pp.131-142. <https://doi.org/10.1016/j.compgeo.2018.12.022>.

Persistent Link:

<http://hdl.handle.net/11343/282932>