Numerical Investigation of Dynamic Installation of Torpedo Anchors in Clay

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ABSTRACT

This paper reports the results from three-dimensional dynamic finite element analysis undertaken to provide insight into the behaviour of torpedo anchors during dynamic installation in non-homogeneous clay. The large deformation finite element (LDFE) analyses were carried out using the coupled Eulerian-Lagrangian approach, modifying the simple elastic-perfectly plastic Tresca soil model to allow strain softening, and incorporate strain-rate dependency of the shear strength using the Herschel-Bulkley model. The results were validated against field data and centrifuge test data prior to undertaking a detailed parametric study, exploring the relevant range of parameters in terms of anchor shaft length and diameter; number, width and length of fins; impact velocity and soil strength.

The anchor velocity profile during penetration in clay showed that the dynamic installation process consisted of two stages: (a) in stage 1, the soil resistance was less than the submerged weight of the anchor and hence the anchor accelerated; (b) in stage 2, at greater penetration, frictional and end bearing resistance dominated and the anchor decelerated. The corresponding soil failure patterns revealed two interesting aspects including (a) mobilization of an end bearing mechanism at the base of the anchor shaft and fins and (b) formation of a cavity above the shaft of the installing anchor and subsequent soil backflow into the cavity depending on the soil undrained shear strength. To predict the embedment depth in the field, an improved rational analytical embedment model, based on strain rate dependent shearing resistance and fluid mechanics drag resistance, was proposed, with the LDFE data used to calibrate the model.

43 KEYWORDS: torpedo anchors; clays; dynamic installation; embedment depth; numerical
44 modelling; offshore engineering

45

46 NOMENCLATURE

47	A_A	anchor shaft cross-section area
48	A_{bF}	fins projected area
49	A_p	anchor shaft and fins projected area
50	A_s	total surface area of anchor
51	A_{sA}	embedded anchor shaft surface area
52	A_{sF}	embedded fin surface area
53	C_d	drag coefficient
54	D_A	anchor shaft diameter
55	D_p	equivalent diameter (including fins)
56	d_t	anchor tip penetration depth
57	$d_{e,t}$	installed anchor tip embedment depth
58	$d_{e,p}$	installed anchor padeye embedment depth
59	E_{total}	total energy during anchor penetration
60	F_b	end bearing resistance
61	$F_{b,bA}$	end bearing resistance at base of anchor shaft
62	$F_{b,bF}$	end bearing resistance at base of anchor fins
63	F_d	inertial drag resistance

64	F_f	frictional resistance
65	F_{fA}	frictional resistance along shaft
66	F_{ff}	frictional resistance along fins
67	F_γ	buoyant weight of soil displaced by anchor
68	h_d	anchor drop height
69	h_{min}	minimum element size
70	k	shear strength gradient with depth
71	L_A	anchor shaft length
72	L_F	fin length
73	L_T	anchor shaft tip length
74	m	anchor mass
75	m'	anchor effective mass
76	$N_{c,bA}$	bearing capacity factor at base of anchor shaft
77	$N_{c,bF}$	bearing capacity factor at base of anchor fins
78	n	factor relating operative shear strain rate to normalized velocity
79	R_f	factor related to effect of strain rate
80	R_{f1}	factor related to effect of strain rate for end bearing resistance
81	R_{f2}	factor related to effect of strain rate for frictional resistance

82	S_t	soil sensitivity
83	S_u	undrained shear strength
84	S_{um}	undisturbed soil strength at mudline
85	$S_{um,ref}$	reference undisturbed soil strength at mudline
86	$S_{u,bA}$	undrained shear strength at bottom of anchor shaft
87	$S_{u,bF}$	undrained shear strength at bottom of fins
88	$S_{u,sA}$	average undrained shear strength over embedded length of shaft
89	$S_{u,sF}$	average undrained shear strength over embedded length of fin
90	$S_{u,ref}$	reference undrained shear strength
91	$S_{u,tip}$	undrained shear strength at anchor tip level
92	t	incremental time
93	t_F	fin thickness
94	v	anchor penetrating velocity
95	v_i	anchor impact velocity
96	w_F	fin width
97	W_d	anchor dry weight
98	W_s	anchor submerged weight in water
99	z	depth below soil surface

100	α	interface friction ratio
101	β	shear-thinning index
102	β_{tip}	anchor tip angle
103	δ_{rem}	fully remoulded ratio
104	η	viscous property
105	λ	rate parameter for logarithmic expression
106	μ	rate parameter for power expression
107	μ_c	coulomb friction coefficient
108	γ'	effective unit weight of soil
109	$\dot{\gamma}$	shear strain rate
110	$\dot{\gamma}_{\text{ref}}$	reference shear strain rate
111	θ_0	pullout angle at the mudline
112	θ_a	pullout angle at the padeye
113	ρ_s	submerged density of soil
114	τ_{max}	limiting shear strength at soil-anchor interface
115	ξ	cumulative plastic shear strain
116	ξ_{95}	cumulative shear strain required for 95% remoulding

1 INTRODUCTION

Dynamically installed anchors (DIAs) are increasingly considered for mooring floating facilities in deep waters. The most commonly used DIA geometry is rocket-shaped, referred to as a torpedo anchor, and may feature up to 4 fins at the trailing edge (see Figure 1). The anchor is released from a specified height above the seabed. This allows the anchor to gain velocity as it falls freely through the water column before impacting the seafloor and embedding into the sediments.

One of the key challenges associated with torpedo anchors includes accurate prediction of the anchor embedment depth. This is complicated by the very high strain rates (exceeding 25 s^{-1}) at the soil anchor interface, resulting from the high penetration velocities; hydrodynamic aspects related to possible entrainment of water adjacent to the anchor and limited understanding of the soil failure mechanisms. The high strain rate leads to increase in undrained shear strength of the soil in the vicinity of the anchor while, in the absence of any better model, a simple limit equilibrium approach comprising frictional and bearing terms is used to estimate penetration resistance.

1.1 Previous work

Investigations on dynamic installation of anchors are very limited, and mostly through centrifuge modelling and field trials, with a summary given by Hossain *et al.* (2014). More recently the problem has been addressed through numerical modelling. Raie and Tassoulas (2009) carried out analysis using a computational fluid dynamics model, modelling the soil as a viscous fluid. Sturm *et al.* (2011) performed quasi-static simulation of the installation process of a torpedo anchor using an implicit finite element code. However, the installation process of a torpedo anchor is a dynamic soil-structure interaction problem involving

extremely large deformations. Nazem *et al.* (2012) carried out dynamic large deformation finite element (LDFE) analysis of a free-falling cone penetrometer (similar to a finless anchor) using an arbitrary Lagrangian-Eulerian (ALE) approach, accounting for strain rate dependency, but considering uniform clay and assuming a frictionless cone-soil interface. As discussed later, frictional resistance along the surfaces of the anchor and the soil strength gradient have significant influences on the anchor embedment depth.

1.2 Theoretical prediction method for anchor embedment depth

Bearing resistance method

Due to the relative scarcity of investigations to assess installation depths of torpedo anchors, current practice relies on the limit equilibrium method proposed by True (1974), based on Newton's second law of motion and considering uniform clay. Several studies (Medeiros, 2002; de Araujo *et al.*, 2004; Brandão *et al.*, 2006; Richardson *et al.*, 2009; O'Loughlin *et al.*, 2009, 2013, Chow *et al.*, 2014; Hossain *et al.*, 2014) have adopted a similar approach, with variations on the inclusion and formulation of the various forces acting on the torpedo anchor. The equations of motion may be expressed as

$$\begin{aligned}
 m \frac{d^2 z}{dt^2} &= W_s - F_\gamma - R_{f1} F_b - R_{f2} F_f - F_d \\
 &= W_s - F_\gamma - R_{f1} (F_{b,bA} + F_{b,bF}) - R_{f2} (F_{fA} + F_{fF}) - F_d \\
 &= W_s - F_\gamma - R_{f1} (N_{c,bA} s_{u,bA} A_A + N_{c,bF} s_{u,bF} A_{bF}) - \alpha R_{f2} (s_{u,sA} A_{sA} + s_{u,sF} A_{sF}) \\
 &\quad - \frac{1}{2} C_d \rho_s A_p v^2
 \end{aligned} \tag{1}$$

The terms used in the above expression are defined under nomenclature. R_{f1} and R_{f2} reflect the effects of shear strain rate for end bearing and frictional resistance, respectively. The frictional resistance term (F_f) comprises friction along the shaft (F_{fA}) and the fins (F_{fF}), while the bearing resistance term (F_b) includes end bearing at the base of the shaft ($F_{b,bA}$) and fins

($F_{b,bF}$). In addition, if soil backflow occurs above the installing anchor, reverse end bearing at the upper end of the shaft ($F_{b,tA}$) and fins ($F_{b,tF}$) must be accounted for. F_γ is the buoyant weight of the displaced soil and F_d is the inertial ‘drag’ resistance generally expressed in terms of a drag coefficient, C_d , as indicated (with ρ_s the submerged soil density and v the penetration velocity).

In practice, natural fine grained soils exhibit strain-rate dependency and also soften as they are sheared and remoulded. For dynamic installation of torpedo anchors, the rate dependency of shear strength will have an important influence on the anchor behaviour, as mapped above field penetrometer test and closure to debris flow by Randolph *et al.* (2007). There is general agreement that the undrained strength increases with increasing shear strain rate; typically the strength enhancement is expressed as either a semi-logarithmic or power law function of the strain rate, although it is acknowledged that these tend to under-predict the increase if extrapolated to very high rates (Biscontin and Pestana, 2001; Chung *et al.*, 2006; DeGroot *et al.*, 2007; Lunne and Andersen, 2007; Low *et al.*, 2008; Lehane *et al.*, 2009; Dejong *et al.*, 2012).

The parameter R_f (both R_{f1} and R_{f2}) may be expressed as

$$R_f = 1 + \lambda \log\left(\frac{\dot{\gamma}}{\dot{\gamma}_{ref}}\right) \quad \text{or} \quad R_f = \left(\frac{\dot{\gamma}}{\dot{\gamma}_{ref}}\right)^\mu \quad \text{with } R_f \geq 1 \quad (2)$$

where λ and μ are strain rate parameters in the respective formulations, $\dot{\gamma}$ is the strain rate and $\dot{\gamma}_{ref}$ is the reference strain rate at which the undrained shear strength was measured. During anchor penetration, the shear strain rate varies through the soil body, but it is reasonable to assume that the net rate parameter may be estimated in terms of an operational

strain rate expressed as some factor, n , times the normalized velocity (v/D_A). Equation 2 can therefore be replaced by

$$R_f = 1 + \lambda \log \left(\frac{nv/D_A}{\dot{\gamma}_{ref}} \right) \quad \text{or} \quad R_f = \left(\frac{nv/D_A}{\dot{\gamma}_{ref}} \right)^\mu \quad \text{with } R_f \geq 1 \quad (3)$$

For bearing type problems the value of n is expected to be close to unity (Zhu and Randolph, 2011), while for frictional resistance it will be an order of magnitude higher (Einav and Randolph, 2006; Steiner *et al.*, 2014; Chow *et al.*, 2014). For anchor installation, the normalized velocity, v/D_A , of $\sim 25 \text{ s}^{-1}$ is respectively about 6 and 2 orders of magnitude higher compared with the shear strain rate in a laboratory simple shear test ($\sim 10^{-5} \text{ s}^{-1}$) and in situ penetrometer (cone, T-bar and ball) tests ($v/D \sim 0.2 \sim 0.5 \text{ s}^{-1}$; Zhou and Randolph, 2009; Boukpeti *et al.*, 2012). For this high velocity problem, a Herschel-Bulkley rheological model (Herschel and Bulkley, 1926) will be more appropriate as discussed later, since it is commonly used for studying the flow of non-Newtonian fluids over several orders of magnitude of strain rate (e.g. Coussot *et al.*, 1998).

Total energy method

Recently, O'Loughlin *et al.* (2013) proposed a simple expression utilising a dataset from centrifuge model tests. Total energy, defined as the sum of the kinetic energy of the anchor at the mudline and the potential energy released as it penetrates the seabed, and soil strength gradient were used to estimate the normalized embedment depth as

$$\frac{d_{e,t}}{D_p} \approx \left(\frac{E_{total}}{kD_p^4} \right)^{1/3} \quad (4)$$

where

$$E_{\text{total}} = \frac{1}{2}mv_i^2 + m'gd_{e,t} \quad (5)$$

with m' being the effective mass of the anchor (submerged in soil), g Earth's gravitational acceleration of 9.81 m/s^2 and D_p the equivalent diameter (including fins).

1.3 Purpose of the Study

The purpose of this paper is to investigate dynamic installation of torpedo anchors, after impacting a non-homogeneous seabed at high velocity, through three-dimensional (3D) dynamic LDFE analyses. Strain softening and strain rate dependency of undrained shear strength are accounted for since both can have a significant effect on embedment. An extensive parametric investigation was undertaken, varying the relevant range of various parameters related to the anchor geometry (and hence mass) and soil strength. An improved rational approach is then proposed for assessing the embedment depth of anchors in the field.

2 NUMERICAL ANALYSIS

2.1 Geometry and parameters

Two main types of DIA geometry are used in the field: (a) torpedo anchor and (b) OMNI-MaxTM anchor. This study has considered a torpedo anchor, consisting of a circular shaft of diameter D_A , to which 0~4 rectangular fins are attached. The anchor penetrates dynamically into a clay deposit with strength increasing linearly with soil depth z , as illustrated schematically in Figure 1. Figure 2 1, where the mudline strength is s_{um} and strength gradient is k . The effective unit weight of the soil, assumed uniform, is γ' . The anchor shaft length is L_A (including tip length, L_T), fin length L_F ($= L_{F1} + L_{F2} + L_{F3}$) and fin width w_F . The shape

was chosen similar to the Petrobras anchors, as illustrated by Medeiros (2002), de Araujo *et al.* (2004) and Brandão *et al.* (2006).

In order to select realistic anchor and soil parameters, a survey was carried out through reported case histories and field trials at various locations in the Campos Basin, offshore Brazil; Vøring Plateau, Troll Field and Gjøa Field in the North Sea, off the Western coast of Norway (Medeiros, 2002; de Araujo *et al.*, 2004; Brandão *et al.*, 2006; Zimmerman *et al.*, 2009; Lieng *et al.*, 2010) and offshore geotechnical characterisation report (Argiolas and Rosas, 2003; Randolph, 2004). Hossain *et al.* (2014) provided a summary of these field tests.

The shafts of torpedo anchors are 0.75~1.2 m in diameter (D_A) and 12~17 m long (L_A), with conical (angle $\beta_{tip} = 30\sim60^\circ$) or ellipsoidal tip. The rectangular (or trapezoidal) fins are 4~11 m long (L_F), 0.45~0.9 m wide (w_F) and 0.05~0.09 m thick (t_F). Dry weight of anchors (W_d) ranges from 23 to 115 tonnes. Anchors are released from a drop height of $h_d = 30\sim150$ m from the seabed, achieving impact velocities v_i of 10~35 m/s, and embedding to depths $d_{e,t}$ of up to 2~3 times their length.

In the regions where DIAs have been used, the seabed sediments consist of clayey deposits with undrained shear strength s_{um} of 2~10 kPa at the mudline increasing with depth with a gradient k of 1~3 kPa/m, measured by means of in situ vane and cone penetration tests and laboratory simple shear tests. The effective unit weight of soil, γ' , typically varies between 5 and 8 kN/m³.

2.2 3D finite element modelling

CEL method for large deformation problem

3D LDFE analyses were carried out using the coupled Eulerian-Lagrangian (CEL) approach in the commercial package ABAQUS/Explicit (Dassault, 2011). Qiu *et al.* (2011), Chen *et al.*

(2013), Tho *et al.* (2013), Hu *et al.* (2014) and Zheng *et al.* (2015) investigated various geotechnical problems using the CEL approach and confirmed its applicability to solve problems involving large deformations.

The CEL approach is identified as advantageous in circumventing mesh distortion in large deformation problems. The soil is tracked as it flows through a Eulerian mesh, fixed in space, by computing the material volume fraction in each element. The Eulerian element can be materially void or occupied partially or fully by more than one material, with the volume fraction representing the portion of that element filled with a specified material. The structural element, such as a torpedo anchor, is discretised with Lagrangian elements, which can move through the Eulerian mesh without resistance until they encounter Eulerian elements containing soil. The interaction between the structure and the soil is represented using a contact algorithm named ‘general contact’ in ABAQUS (Dassault, 2011).

Mesh and boundary conditions

Considering the symmetry of the problem, only a quarter anchor and soil were modelled. The radius and height of the soil domain were $40D_A$ ($\sim 32D_p$ for 4-fin anchor) and $\sim 8L_A$, respectively, to ensure that the soil extensions are sufficiently large to avoid boundary effect in dynamic analyses. A typical mesh is shown in Figure 2. The mesh comprised 8-noded linear brick elements with reduced integration, and a fine mesh zone is generated to accommodate the anchor trajectory during the entire installation. A 5 m thick void (i.e. material free) layer was set above the soil surface (see Figure 2c), allowing the soil to heave by flowing into the empty Eulerian elements during the penetration process. The anchor was simplified as a rigid body and assumed to remain vertical during the penetration process, with no tilt. The anchor dynamic installation was modelled from the soil surface, with a given velocity v_i .

269 ***Constitutive law and material parameters***

270 The installation of torpedo anchors in clay is completed under nearly undrained conditions.
 271 The soil was thus modelled as an elasto-perfectly plastic material obeying a Tresca yield
 272 criterion, but extended as described later to capture strain-rate and strain-softening effects. A
 273 user subroutine was implemented to track the evolving soil strength profile. The elastic
 274 behaviour was defined by a Poisson's ratio of 0.49 and Young's modulus of $500s_u$ throughout
 275 the soil profile. A uniform submerged unit weight of 6 kN/m^3 was adopted over the soil depth,
 276 representing a typical average value for field conditions.

277 ***Incorporation of combined effects of strain rate and strain softening***

278 The Tresca soil model was extended in order to consider the combined effects of rate
 279 dependency and strain softening. For rate dependency, a unified formulation, originating
 280 from the Herschel–Bulkley model (Herschel and Bulkley, 1926), is used for simulating
 281 geotechnical problems involving high strain rate. It can be adopted to characterise the
 282 strength behaviour of both a soil material and also a fluid-like material (Zhu and Randolph,
 283 2011; Boukpeti *et al.*, 2012). The undrained shear strength at individual Gauss points was
 284 modified immediately prior to re-meshing, according to the average rate of maximum shear
 285 strain in the previous time step and the current accumulated absolute plastic shear strain,
 286 according to

$$287 \quad s_u = \left[1 + \eta \left(\frac{\dot{\gamma}}{\dot{\gamma}_{\text{ref}}} \right)^\beta \right] \left[\delta_{\text{rem}} + (1 - \delta_{\text{rem}}) e^{-3\zeta/\zeta_{95}} \right] \frac{s_{u,\text{ref}}}{(1 + \eta)} \quad (6)$$

288 where $s_{u,\text{ref}}$ is the shear strength at the reference shear strain rate of $\dot{\gamma}_{\text{ref}}$. The first bracketed
 289 term of Equation 6 augments the strength according to the operative shear strain rate, $\dot{\gamma}$,
 290 relative to a reference value, $\dot{\gamma}_{\text{ref}}$, which is typically around 10^{-5} s^{-1} for laboratory element

tests and up to $\sim 0.5 \text{ s}^{-1}$ for field penetrometer testing (although in the latter case the high strain rate is partly compensated for by strain softening (Zhou and Randolph, 2009)). Ideally, the shear strength should be deduced from a reference strain rate, $\dot{\gamma}_{\text{ref}}$, that is relatively close (within 2 to 3 orders of magnitude) to that relevant for the application. Here a value of 0.1 s^{-1} was adopted. The shear strain rate, $\dot{\gamma}$, within the soil was evaluated according to

$$\dot{\gamma} = \frac{\Delta \epsilon_1 - \Delta \epsilon_3}{\Delta t} \quad (7)$$

where $\Delta \epsilon_1$ and $\Delta \epsilon_3$ are the cumulative major and minor principal strains, respectively, over the incremental time period, Δt . The parameter η is a viscous property and β the shear-thinning index. In clay soil, typical values of η and β are in the range of $0.1 \sim 2.0$ and $0.05 \sim 0.15$, respectively (Boukpeti *et al.*, 2012).

The second part of Equation 6 models the degradation of strength according to an exponential function of cumulative plastic shear strain, ξ , from the intact condition to a fully remoulded ratio, δ_{rem} (the inverse of the sensitivity, S_t). The relative ductility is controlled by the parameter, ξ_{95} , which represents the cumulative shear strain required for 95% remoulding. Typical values of ξ_{95} have been estimated as around $10 \sim 30$ (i.e. $1,000 \sim 3,000\%$ shear strain; Randolph, 2004; Zhou and Randolph, 2009).

The soil-anchor interface was modelled as frictional contact, using a general contact algorithm and specifying a (total stress) Coulomb friction law together with a limiting shear stress (τ_{max}) along the anchor-soil interface. The Coulomb friction coefficient was set to a high value of $\mu_c = 50$, in order to allow the value of τ_{max} to govern failure. Within the CEL, the value of the limiting interface friction must be set prior to the analysis, before the value of the ‘adjacent’ soil strength is known. To overcome this difficulty, for each case, the limiting interface friction was determined by: (1) simulating anchor penetration with frictionless

contact; (2) obtaining the final anchor tip penetration depth and calculating $s_{u,ref}$ at that depth; and (3) setting τ_{max} equal to an interface friction ratio, α , times the calculated $s_{u,ref}$ at the final tip depth (from frictionless contact), with α taken as the inverse soil sensitivity, $1/S_t$. Due to the limitation of the current CEL approach, τ_{max} is a constant value on all the anchor surfaces during the entire calculation. At shallow depth, where τ_{max} may exceed the rate-dependent shear strength of the adjacent soil, failure may occur in the adjacent soil, rather than at the interface. The contact interface is created between Lagrangian mesh and Eulerian material, and automatically computed and tracked during the analysis. It should be noted that the limiting interface friction is independent of the shearing rate, according to what is possible in CEL.

3 VALIDATION AGAINST CENTRIFUGE TEST DATA AND FIELD DATA

3.1 Optimized Mesh Density

A very fine soil mesh was necessary to capture the anchor-soil contact accurately. Therefore, mesh convergence studies were first performed to ensure that the mesh was sufficiently fine to give accurate results. As discussed later, the soil parameters were taken as $s_{u,ref} = 5 + 2z$ kPa, $\dot{\gamma}_{ref} = 0.1 \text{ s}^{-1}$, $\eta = 1.0$, $\beta = 0.1$, $\delta_{rem} = 1/S_t = 1/3$ and $\xi_{95} = 20$, and a 4-fin anchor was chosen with $D_A = 1.07 \text{ m}$, $L_A = 17 \text{ m}$, $L_F = 10 \text{ m}$, $w_F = 0.9 \text{ m}$. Five different mesh densities were considered (see Table 1). The time-penetration curves obtained based on each of these meshes are shown in Figure 3. The numerical results based on mesh 4 and mesh 5, with minimum element sizes (h_{min}) $0.019D_A$ and $0.014D_A$ respectively, are essentially identical, indicating that mesh convergence was achieved with the density of mesh 4. As such, for subsequent parametric analyses, the typical minimum soil element size along the trajectory of

the anchor is selected as $0.019D_A$. With this size of element, typical computation times on a PC with 10 CPU cores (frequency of 3.20 GHz) were about 40 ~ 50 h for a torpedo anchor installation.

3.2 Comparison between LDFE Result and Centrifuge Test Data

The LDFE results were validated against centrifuge test data and field data. Hossain *et al.* (2014) presented data from a centrifuge test carried out at 200 g in kaolin clay. The model anchor, machined from brass (density 8400 kg/m^3) with geometry similar to that considered in this study ($D_A = 1.2 \text{ m}$, $L_A = 15 \text{ m}$, $L_F = 10 \text{ m}$, $w_F = 0.9 \text{ m}$) was dropped dynamically ($v_i = 14.94 \text{ m/s}$). The soil undrained shear strength of $s_{u,\text{ref}} = 1 + 0.85z \text{ kPa}$ was deduced from T-bar penetration tests. In the LDFE simulation, these parameters and $\eta = 1.0$; $\beta = 0.1$; $\delta_{\text{rem}} = 1/S_t = 1/2.5$; $\xi_{95} = 20$; and $\dot{\gamma}_{\text{ref}} = 0.1 \text{ s}^{-1}$ were used.

Figure 4a shows the penetration profiles from the centrifuge test and LDFE analysis. Good agreement can be seen, with the computed anchor tip embedment depth only 1.4% greater than that measured.

3.3 Comparison between LDFE Results and Field Test Data

Medeiros (2002) reported data from a field trial carried out in the Campos basin. A finless anchor ($W_d = 400 \text{ kN}$; $D_A = 0.762 \text{ m}$; $L_A = 12 \text{ m}$) was released from a drop height of $h_d = 30 \text{ m}$, which led to an embedment depth of $d_{e,t} = 20 \text{ m}$. LDFE analysis was carried out using these data, $v_i = 22 \text{ m/s}$ (Raie and Tassoulas, 2009) and $\eta = 1.0$; $\beta = 0.1$; $\dot{\gamma}_{\text{ref}} = 0.1 \text{ s}^{-1}$; $\delta_{\text{rem}} = 1/S_t = 1/3$; $\xi_{95} = 20$; $\dot{\gamma}_{\text{ref}} = 0.1 \text{ s}^{-1}$. The undrained shear strength ($s_{u,\text{ref}} = 5 + 2z \text{ kPa}$) was measured by in situ vane tests and cone penetration tests. Figure 4b shows the measured and computed embedment depths, with good agreement obtained.

Validation was also undertaken against field data on a 4-fin torpedo anchor, as reported by Brandão *et al.* (2006). Eighteen torpedo anchors ($W_d = 961$ kN, $D_A = 1.07$ m, $L_A = 17$ m, $L_F = 10$ m and $w_F = 0.9$ m) were installed in a soft normally consolidated clay deposit ($s_{u,ref} \approx 5 + 2z$ kPa) in the Campos basin. The achieved impact velocity was $v_i \approx 26.8$ m/s and the average penetrations were about 35.2 m (see Figure 4b). The profile from corresponding LDFE analysis is included in Figure 4b for comparison. Excellent agreement can be seen not only in terms of the final embedment depth but also the full penetration profile.

These validation analyses have confirmed the capability and accuracy of the CEL approach in assessing the embedment depth during dynamic installation of torpedo anchors in non-homogeneous clay.

4 RESULTS AND DISCUSSION: PARAMETRIC STUDY

To examine the effect of various factors on the embedment depth, an extensive parametric study was carried out varying (a) the anchor geometry: diameter (0.8 ~ 1.2 m) and length of shaft (12.5 ~ 15 m); (b) the fin configuration: number (0 ~ 4), length ($L_F = 5$ and 10 m) and width ($w_F = 0.45$ and 0.9 m); (c) the impact velocity ($v_i = 15 \sim 35$ m/s); and (d) the soil undrained shear strength ($s_{u,ref} = 1 + 1z$, $5 + 2z$ kPa and $10 + 3z$ kPa). The results from this parametric study, as assembled in Table 2, are discussed below. Parameters in terms of rate dependency and strain-softening were taken as $\eta = 1.0$; $\beta = 0.1$; $\dot{\gamma}_{ref} = 0.1$ s⁻¹; $\delta_{rem} = 1/3$; and $\xi_{95} = 20$, as they typically provided excellent match with the field and centrifuge tests data. More details of the influence of these parameters have also been investigated and reported in Kim et al. (2015).

4.1 Anchor penetration profile and soil failure mechanisms

Figure 5 depicts the instantaneous (resultant) velocity vectors during penetration of anchors with and without fins ($L_A = 17$ m, $D_A = 1.07$ m, $\beta_{tip} = 30^\circ$, $v_i = 20$ m/s; in Groups I and III, Table 2) in soft clay with $s_{u,ref} = 5 + 2z$ kPa (note, the instantaneous velocity vector plots are not the same as for the real Lagrangian material, but the representation of the deformed soil flow in Eulerian element; e.g. Tho *et al.*, 2013). Because of autoscaling, the regions of red vectors extend out to a velocity of approximately 0.5% of the current anchor velocity. It shows the soil failure mechanisms at 4 different time intervals (and penetrations) after impacting the clay surface. The anchor velocity profile and corresponding soil failure mechanisms can be divided into main two stages. Stage 1 corresponds to shallow penetration where the anchor accelerates although it advances into the soil. In this stage, (a) at $t = 0.1$ s (velocity = ~ 20.7 m/s, penetration ~ 2 m), the mechanism illustrates the pattern immediately after impacting the seafloor; (b) at $t = 0.3$ s (velocity = ~ 22 m/s, penetration ~ 6 m), significant soil movement occurs adjacent to the embedded anchor; (c) at $t = 0.6$ s (velocity = ~ 23.0 m/s, penetration ~ 13 m), the soil incremental displacement is concentrated at the tip of the anchor and base of the fins, indicating that shearing within the soil body is confined to the end bearing mechanisms. At that stage, the embedment depths for the 4-fin ($L_F = 10$ m, $w_F = 0.9$ m, $W_s = 850$ kN) and finless ($W_s = 708$ kN) anchors are very similar. The (minimal) effect of the $\sim 20\%$ greater submerged anchor weight of the 4-fin anchor was counterbalanced by the increasing soil resistance on the fins, resulting in the demarcation point between stage 1 and stage 2 (i.e. the depth of stage 1) similar to the finless anchor.

In Stage 2 (referring to penetration after attaining maximum velocity), the frictional resistance (F_s) and end bearing (F_b) overcome the submerged weight, negated by the buoyancy force, of the anchor ($W_s - F_\gamma$) and the anchor decelerates (see Equation 1). Finally,

the velocities in the soil decrease towards zero as the anchor reaches its final embedment depth, coming to rest at similar times ($t \sim 2.0$ s) at penetrations of 41.9 m and 32.1 m respectively for the finless and 4-fin anchor (see Figure 5). The deceleration of the 4-fin anchor is much greater than that of the finless anchor for this stage because of the frictional and end-bearing resistance of the fins.

4.2 Embedment depth $d_{e,t}$

In order to demonstrate the effect of the factors noted at the outset of this section, the tip embedment depths, $d_{e,t}$, are presented as a function of impact velocity, v_i , in Figure 6 and Table 2. Note, Figure 6 also includes the anchor penetration depths at the end of stage 1. The numerical results indicate that the embedment depth increases somewhat linearly with the impact velocity. For the anchor tip penetration depths at the end of stage 1, the reverse (although marginal) trend is true, but, overall, the effect of the considered factors is trivial and hence corresponding discussions are not included in the following sub-sections.

Effect of number, length and width of fins

The anchors with 4-fins and 2-fins and finless anchors (Group I ~ III in Table 2) were analyzed to explore the effect of the number of fins. From Figure 6, it can be seen that the embedment depth increases significantly with reducing number of fins. For instance, for $v_i = 15$ m/s in Table 2, the embedment depth $d_{e,t}$ increases from 29.0 to 36.8 m (i.e. 27%) as the number of fins reduces from 4 to 0. This is due to the combined effects of reducing fin area for frictional resistance and end bearing, and also anchor weight. This gap widens as the impact velocity increases, and becomes somewhat constant when v_i exceeds 25 m/s.

Similarly, analyses were carried out halving the width (w_F) and length (L_F) of fins (Groups IV and V, Table 2). The embedment depth increases by 8~9% with halving L_F , while it increases by 4.5~5.1% with halving w_F . Compared to the base case, for the anchor with half

L_F , A_{bF} is identical, but the relevant (average) shear strengths $s_{u,bF}$ and $s_{u,Fs}$ are significantly lower. The reverse is true for the anchor with half w_F . This is illustrated further in Figure 7, depicting instantaneous velocity plots for various anchor configurations.

Effect of length and diameter of shaft (L_A , D_A)

The effect of the length of the anchor shaft (L_A) was investigated, reducing the length to $L_A = 15$ m, but keeping the other parameters such as length of fins ($L_F = 10$ m) constant (Group VI, Table 2). The embedment depth reduces by 13.7% for $v_i = 20$ m/s, and by about 11% for $v_i \geq 25$ m/s. Note, the corresponding reduction in surface area is 5.5% and that in submerged weight is 10%. While these have compensating effects, it seems that the reduction in weight (and hence v_{max}) is a critical factor. In addition, as apparent from Figure 7d, as the distance between the base of the shaft and fins reduces, the interaction between the corresponding end bearing mechanisms increases, potentially increasing strain rates and hence soil strength.

Analyses were also carried out varying D_A to 0.8 m and 1.2 m to examine the effect of the anchor shaft diameter (Group VII, Table 2). These not only change the surface area A_s (-10.7%, +5.1%) and weight (-37.3%, +22.0%) but also the projected area A_p (-33.6%, +19.3%). Interestingly, the embedment depth decreases by 9% with reduced D_A . Former cases showed that the anchor embedment depth is influenced strongly by the surface and projected areas. It seems though that the lower weight (37.3% lower than the base case) has more than compensated the reduction of frictional and bearing resistance.

Effect of soil undrained shear strength

The undrained shear strength of the soil was thus far kept constant as $s_{u,ref} = 5 + 2z$ kPa. The effect of soil undrained shear strength was explored considering a lower $s_{u,ref}$ of $1 + 1z$ kPa and higher $s_{u,ref}$ of $10 + 3z$ kPa (Groups VIII and IX, Table 2). The embedment depths are given in Table 2. The time-penetration profiles for the 4-fin ($v_i = 20$ m/s) and finless ($v_i = 15$

m/s) anchors are plotted in Figure 8a and b, respectively. All profiles are consistent up to a penetration of ~15 m, with maximum velocities reached of 22.2 ~ 24.4 m/s for the 4-fin and 17.8 ~ 20.97 m/s for the finless anchor. Again, this confirms that, at shallow penetration (Stage 1), the end bearing and frictional resistance (and hence the soil strength) have a limited influence. Beyond that depth, with the domination of bearing and frictional resistance, the responses depart from each other with the increased shear strength resulting in a shallower embedment depth and vice versa.

For lower impact velocities, the embedment depth reduces significantly (15~27%) with increasing shear strength. This gap reduces (2.3~7.5%) as the impact velocity increases from 20 m/s to 30 m/s. This influence is of course more pronounced for the 4-fin anchor with greater frictional and bearing areas compared to the finless anchor.

4.3 Cavity Condition after Penetrating: Effect of undrained shear strength

The effect of the undrained shear strength on the cavity depth is shown in Figure 9 illustrating the top and side view of anchor installation. Although the hole above the penetrating anchor never completely closes, it can be seen that (a) for clay with highest $\sum_{m,ref}/\gamma'D_A = 1.56$ (and lowest $kD_A/\sum_{m,ref} = 0.32$; Figure 9a), the cavity formed above the penetrating anchor remains open and the soil surface heaves slightly; (b) for clay with lowest $\sum_{m,ref}/\gamma'D_A = 0.16$ (and highest $kD_A/\sum_{m,ref} = 1.07$; Figure 9b), the anchor becomes essentially fully covered by the backfilled soil or nearly replenished immediately after the anchor top penetrates below the soil surface; and (c) for clay with medium $\sum_{m,ref}/\gamma'D_A = 0.78$ (and medium $kD_A/\sum_{m,ref} = 0.43$; see Figure 9c), the soil partly flows back into the cavity, but the cavity remains open (with distorted wall) down to the top of the anchor. From centrifuge model tests in clay, Hossain *et al.* (2014) also observed a fully replenished cavity for $\sum_{m,ref}/\gamma'D_A = 0.12$, $kD_A/\sum_{m,ref} = 1.02$.

5 PREDICTION OF EMBEDMENT DEPTH IN FIELD

5.1 Bearing Resistance Method

Equation 1 can be used to assess anchor embedment depth in the field, with the R_f term calculated as

$$R_{f1} = \left[1 + \eta \left(\frac{nv/D_A}{\dot{\gamma}_{ref}} \right)^\beta \right] \frac{1}{(1 + \eta)} \quad (8)$$

A suggested procedure is outlined here.

1. Determine representative values of the soil parameters $\Sigma_{m,ref}$, k , and effective self-weight, γ' .
2. For the given anchor geometry calculate A_A , A_{bF} , A_{sA} , A_{sF} , A_p .
3. Take $C_d = 0.63$ for 4-fin anchors and 0.24 for finless anchors (Lieng *et al.*, 1999; 2000; O'Loughlin *et al.*, 2013), v at the mudline = v_i , $\alpha = 1/S_t$.
4. Adopt appropriate values for the bearing capacity factors $N_{c,bA}$ and $N_{c,bF}$ accounting for the geometry and the softening or sensitivity of the soil; alternatively, deep bearing factors of cone and strip footing of 13.6 (Low *et al.*, 2010) and 7.5 (Skempton, 1951) can be considered for $N_{c,bA}$ and $N_{c,bF}$, respectively due to the geometrical similarity noting that the effect of softening is small for torpedo anchor installation (Kim *et al.*, 2015).
5. For clay with low $\Sigma_{m,ref}/\gamma'D_A < \sim 0.2$ (and high $kD_A/\Sigma_{m,ref} > 1$), take account of reverse end bearing at the top of the shaft and fins.
6. Calculate R_{f1} for end bearing using Equation 8 or 3, taking rate parameters for end bearing resistance ($\eta = 1.0$, $\beta = 0.1$ or $\lambda = 0.1$ or $\mu = 0.05$, which correspond to

average values from the typical ranges reported from previous studies¹), $n = 1$ and $\dot{\gamma}_{\text{ref}}$ at which $S_{u,\text{ref}}$ was measured.

7. Take R_{f2} for frictional resistance as $\sim 2R_{f1}$ (Einav and Randolph, 2006; Steiner *et al.*, 2014; Chow *et al.*, 2014).

The contributions of each anchor resistance force are plotted in Figures 10a and 10b (considering $N_{c,bA} = 13.6$, $N_{c,bF} = 7.5$). Note, for these cases with no hole closure for both the finless and 4-fin anchors (see Figure 9c), soil buoyancy increases with depth (see also figures 8b, c and d of O’Loughlin *et al.*, 2013). The figure confirms that the soil resistance at relatively shallow depths (Stage 1, acceleration stage) does not exceed the submerged weight of the anchor ($W_s = 850$ kN for the 4-fin and 708 kN for the finless anchor).

5.2 Modified Energy Method

Figure 11a shows that the computed data in $E_{\text{total}}/kD_p^4 - d_{e,t}/D_p$ space is scattered and Equation 4 underestimates the computed embedment depths. As such, the normalized depths are plotted in Figure 11b as a function of $E_{\text{total}}/kA_sD_p^2$ accounting for the contribution of the total surface area of the anchor ($A_s = A_{sA} + A_{sF}$). All data show a unique trend and can be represented as

¹ The typical ranges of λ (0.05~0.2), μ (0.04~0.1), and β (0.05~0.2) were from the previous studies on field and centrifuge T-bar, ball, shear vane and dynamic piezocone tests in various clays (Biscontin and Pestana, 2001; Chung *et al.*, 2006; Boylan *et al.*, 2007; Peuchen and Mayne, 2007; Low *et al.*, 2008; Lehane *et al.*, 2009; Steiner *et al.*, 2014; Chow *et al.*, 2014) and on high velocity submarine landslides (Boukpeti *et al.*, 2012).

$$\frac{d_{e,t}}{D_p} = 4.0 \left(\frac{E_{total}}{kA_s D_p^2} \right)^{0.389} \quad (9)$$

with correlation coefficient $R^2 = 0.98$ and standard deviation of $(d_{e,t})_{predicted}/(d_{e,t})_{FE}$ of 0.051. This is a simple approach with the only inputs required being the anchor mass and geometry and soil strength gradient.

5.3 Example Application

As discussed previously, Brandão *et al.* (2006) reported field data for 4-fin anchors ($W_s = 850$ kN, $D_A = 1.07$ m, $L_A = 17$ m, $L_F = 10$ m and $w_F = 0.9$ m) dynamically installed ($v_i \sim 26.8$ m/s) in a soft normally consolidated clay deposit ($s_{u,ref} \sim 5 + 2z$ kPa). The embedment depth was calculated using the proposed methods just discussed, and the results are summarized in Table 3. All the proposed equations provide reasonable estimates with an error of -3.2~+3.0% ('+ve' means overestimation) except Equation 4, which under-estimates the penetration depth by 20.1% (as indeed noted by O'Loughlin *et al.*, 2013).

6 CONCLUDING REMARKS

Dynamic installation of torpedo anchors was investigated extensively through 3D dynamic large deformation finite element analyses. The embedment depth of dynamically penetrating torpedo anchors was found to be a function of the impact velocity and geometric parameters of torpedo anchors such as the length and diameter of the shaft; number, width and length of fins; the undrained shear strength of seabed sediments.

The anchor velocity profile during the dynamic installation process consisted of two stages: (a) in stage 1, the anchor resistance was less than the weight of the anchor and hence the anchor accelerated; (b) in stage 2, at greater penetration, frictional and end bearing resistance

dominated and the anchor decelerated. The corresponding soil failure patterns revealed two interesting aspects including (a) mobilization of an end bearing mechanism at the base of the anchor shaft and fins and (b) formation of a cavity above the shaft of the installing anchor and subsequent soil backflow into the cavity depending on the soil undrained shear strength.

For assessing the dynamic embedment depth of torpedo anchors, two models were proposed. Bearing resistance model combines the contribution from anchor submerged weight, end bearing and frictional resistance, and drag inertia force. The soil undrained shear strength was augmented accounting for the effect of high strain rate through either a semi-logarithmic, power or unified Hershel-Buckley rate law, with values suggested for the corresponding rate parameters. A simple total energy approach, modified from that proposed by O'Loughlin *et al.* (2013), was also proposed, taking into account the effect of anchor mass, impact velocity, surface area, projected area equivalent diameter of the anchor and the gradient of the soil undrained shear strength.

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Table 1. Mesh convergence tests

Mesh	Number of elements	$h_{\min}/D_A$
1	1,328,780	0.093
2	2,403,180	0.047
3	3,564,875	0.033
4	7,522,380	0.019
5	11,550,195	0.014

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Table 2. Summary of 3D LDFE analyses performed

Group	Length of anchor shaft, L_A : m	Diameter of anchor shaft, D_A : m	Number of Fins	Length of fins, L_F : m	Width of fins, w_F : m	Submerged anchor weight, W_s (kN)	v_i : m/s	$S_{u,ref}$: kPa	Depth of tip embedment, $d_{e,t}$: m	Notes
I	17	1.07	4	10	0.9	850	15	5 + 2z	29.02	Petrobras T-98 anchor (4 fins)
							20		32.16	
							25		34.91	
							30		38.41	
							35		41.85	
II	17	1.07	2	10	0.9	779	20		36.1	Effect of number of fins (2 fins)
							25		39.36	
							30		43.63	
III	17	1.07	0	-	-	708	15		36.85	Effect of number of fins (finless)
							20		41.91	
							25		46.82	
							30		50.92	
							35		55.76	
IV	17	1.07	4	5	0.9	777	20		35.10	Effect of fin length
							25		37.71	
							30		41.33	
V	17	1.07	4	10	0.45	777	20		33.81	Effect of fin width
							25		36.55	
							30		40.13	
VI	15	1.07	4	10	0.9	759	20		27.73	Effect of anchor shaft
							25		30.97	
							30		34.02	
VII	17	0.8	4	10	0.9	537	20		29.15	Effect of anchor diameter
	17	1.2				1032	25		32.56	
							30		35.55	
							20		37.12	
							25		41.32	
30	44.85									
VIII	17	1.07	4	10	0.9	850	15	1 + 1z	40.10	Effect of soil strength (4 fins)
							20		43.44	
							30		50.10	
							10 + 3z	15	23.96	
								20	26.87	
								30	33.11	
IX	17	1.07	0	-	-	779	15	1 + 1z	57.18	Effect of soil strength (finless)
							20		62.32	
							10 + 3z	15	28.48	
								20	33.27	

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Table 3. Exercise results with present approaches

Anchor	$S_{u,ref}$: kPa	v_i : m/s	Design approach	Rate parameters		$d_{e,t}$: m		Error: %
				Parameter	Value	Measured	Calculated	
T-98 (4-fin) $D_A = 1.08$ m $L_A = 17$ m $L_F = 10$ m $w_F = 0.9$ m $t_F = 0.1$ m $W_s = 850$ kN	$5 + 2z$	26.8	Eq. 1	λ	0.1	35.2	36.3	+3.0
				μ	0.05		34.7	-1.4
				η, β	1.0, 0.1		34.1	-3.2
			Eq. 4	-	-		29.3	-20.1
			Eq. 9	-	-		34.2	-2.9

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687 **No. of Figure: 11**

688 **Figure 1.** Schematic of installed torpedo anchor in clay

689 **Figure 2.** Typical mesh used in CEL analysis: (a) Typical 3D mesh; (b) Detail plan view;
690 (c) Side view

691 **Figure 3.** Effect of FE mesh density on anchor embedment depth ($L_A = 15$ m, $D_A = 1.2$
692 m, $L_F = 10$ m, $w_F = 0.9$)

693 **Figure 4.** Comparison between LDFE result and measured data: (a) Comparison with
694 centrifuge data; (b) Comparison with field data

695 **Figure 5.** Instantaneous velocity plot with embedment ($L_A = 17$ m, $D_A = 1.07$ m, $L_F = 10$
696 m, $w_F = 0.9$ m; in Groups I and III, Table 2)

697 **Figure 6.** Effect of various factors on anchor tip embedment depth (Groups I~VIII,
698 Table 2)

699 **Figure 7.** Instantaneous velocity plot for different anchor fin geometries ($L_A = 17$ m, D_A
700 $= 1.07$ m, $S_{u,ref} = 5 + 2z$ kPa, $v_i = 20$ m/s ; in Groups I, IV, V and VI, Table 2):
701 (a) T-98 anchor; (b) Reduced fin width; (c) Reduced fin length; (d) Reduced
702 anchor shaft

703 **Figure 8.** Effect of soil undrained shear strength on anchor embedment depth (in Groups
704 VIII and IX, Table 2): (a) 4-fin ($L_A = 17$ m, $D_A = 1.07$ m, $L_F = 10$ m, $w_F = 0.9$
705 m); (b) finless ($L_A = 17$ m, $D_A = 1.07$ m)

706 **Figure 9.** Cavity condition above installed anchors ($L_A = 17.0$ m, $D_A = 1.07$ m; in
707 Groups I, III, VIII and IX, Table 2): (a) $S_{u,ref}/\gamma'D_A = 1.56$, $kD_A/S_{u,ref} = 0.32$;
708 (b) $S_{u,ref}/\gamma'D_A = 0.16$, $kD_A/S_{u,ref} = 1.07$; (c) $S_{u,ref}/\gamma'D_A = 0.43$, $kD_A/S_{u,ref} =$
709 0.78

710 **Figure 10.** Contribution of various resistance force components (in Groups I and III,
711 Table 2): (a) 4-fin ($L_A = 17$ m, $D_A = 1.07$ m, $L_F = 10$ m, $w_F = 0.9$ m); (b)
712 finless ($L_A = 17$ m, $D_A = 1.07$ m)

713 **Figure 11.** New methods for assessing anchor embedment depths: (a) Total energy
714 method (O'Loughlin *et al.*, 2013); (b) Modified total energy method

715

Figure 1. Schematic of installed torpedo anchor in clay

721

(a) Typical 3D mesh

(b) Detail plan view

(c) Side view

Figure 2. Typical mesh used in CEL analysis

722

Figure 3. Effect of FE mesh density on anchor embedment depth ($L_A = 15$ m, $D_A = 1.2$ m, $L_F = 10$ m, $w_F = 0.9$)

(a) Comparison with centrifuge data

(b) Comparison with field data

Figure 4. Comparison between LDFE result and measured data

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(a) 4-fin ($L_A = 17$ m, $D_A = 1.07$ m, $L_F = 10$ m, $w_F = 0.9$ m) (b) Finless ($L_A = 17$ m, $D_A = 1.07$ m)**Figure 8. Effect of soil undrained shear strength on anchor embedment depth (in Groups VIII and IX, Table 2)**

Figure 9. Cavity condition above installed anchors ($L_A = 17.0$ m, $D_A = 1.07$ m; in Groups I, III, VIII and IX Table 2)

(a) 4-fin ($L_A = 17 \text{ m}$, $D_A = 1.07 \text{ m}$, $L_F = 10 \text{ m}$, $w_F = 0.9 \text{ m}$) (b) Finless ($L_A = 17 \text{ m}$, $D_A = 1.07 \text{ m}$)**Figure 10. Contribution of various resistance force components (in Groups I and III, Table 2)**

(a) Total energy method (O'Loughlin *et al.*, 2013)

(b) Modified total energy method

Figure 11. New methods for assessing anchor embedment depths