Effect of Strain Rate and Strain Softening on Embedment Depth of a Torpedo Anchor in Clay

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Abstract: Torpedo anchors (of diameter ~1 m) are released from a height of 50~100 m from the seabed, achieving velocities up to 35 m/s at impacting the sediment. The strain rates induced in the surrounding soil by this dynamic installation is therefore significantly higher than those associated with installation of other offshore foundations and anchoring systems. The high strain rates enhance the mobilised undrained shear strength compared to that measured by in-situ penetrometer or laboratory tests. This paper reports the results from dynamic installation of a torpedo anchor in strain softening, rate dependent soft clays, quantifying the effects relative to results for ideal Tresca material. The three-dimensional dynamic large deformation finite element (LDFE) analyses were carried out using the coupled Eulerian-Lagrangian approach. The simple elastic-perfectly plastic Tresca soil model was modified to allow strain softening and strain rate dependency of the shear strength. Parametric analyses were undertaken varying the strain rate parameter, the sensitivity and ductility of the soil, and the soil undrained shear strength. Overall, embedment depth for rate dependent, strain softening clays lay below that for ideal Tresca material. Increased strain rate dependency of the soil led to marked reduction in embedment depth, only partly compensated by brittleness. Key results have been presented in the form of design charts, fitted by simple expressions to estimate the embedment depth of a torpedo anchor.

KEYWORDS: torpedo anchors; clay sensitivity; strain rate dependency; dynamic installation; embedment depth; numerical modelling

42 **Nomenclature**

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

60	F_{fA}	frictional resistance along shaft
61	F_{fF}	frictional resistance along fins
62	F_γ	buoyant weight of soil displaced by anchor (calculated using effective unit weight of
63	soil)	
64	g	earth's gravitational acceleration
65	k	shear strength gradient with depth
66	L_A	anchor shaft length
67	L_F	fin length
68	L_T	anchor shaft tip length
69	m	anchor mass
70	m'	anchor effective mass
71	$N_{c,bA}$	anchor tip bearing capacity factor
72	$N_{c,bF}$	fin bearing capacity factor
73	n	factor relating operative shear strain rate to normalised velocity
74	R_a	average strain rate coefficient for embedment prediction
75	R_b	average strain rate coefficient for energy method
76	R_{f1}	factor related to effect of strain rate and softening for end bearing resistance
77	R_{f2}	factor related to effect of strain rate and softening for frictional resistance

78	S_t	soil sensitivity
79	s_u	undrained shear strength
80	$s_{u,bA}$	undrained shear strength at bottom of anchor shaft
81	$s_{u,bF}$	undrained shear strength at bottom of fins
82	$s_{u,ref}$	reference undrained shear strength
83	$s_{u,sA}$	average undrained shear strength over embedded length of shaft
84	$s_{u,sF}$	average undrained shear strength over embedded length of fin
85	$s_{u,tip}$	undrained shear strength at anchor tip level
86	s_{um}	undisturbed soil strength at mudline
87	$s_{um,ref}$	reference undisturbed soil strength at mudline
88	t	time after anchor tip impacting seabed
89	t_F	fin thickness
90	v	anchor penetration velocity
91	v_i	anchor impact velocity
92	w_F	fin width
93	W_d	anchor dry weight
94	W_s	anchor submerged weight in water
95	z	depth below soil surface

96	α	interface friction ratio
97	β	shear-thinning index
98	β_{tip}	anchor tip angle
99	Δt	incremental time
100	$\Delta \varepsilon_1$	cumulative major principal strain
101	$\Delta \varepsilon_3$	cumulative minor principal strain
102	δ_{rem}	fully remoulded ratio
103	γ'	effective unit weight of soil
104	$\dot{\gamma}_{\text{ref}}$	reference shear strain rate
105	$\dot{\gamma}$	shear strain rate
106	η	viscous property
107	μ_c	Coulomb friction coefficient
108	θ_0	pullout angle at mudline
109	θ_a	pullout angle at padeye
110	ρ_s	submerged soil density
111	τ_{max}	limiting shear strength at soil-anchor interface
112	ξ	cumulative plastic shear strain

- 113 ξ_a average cumulative plastic shear strain for embedment prediction
- 114 ξ_b average cumulative plastic shear strain for energy method
- 115 ξ_{95} cumulative plastic shear strain required for 95% remoulding

Introduction

Dynamically installed anchors (DIAs) are the most recent generation of anchoring systems for mooring floating facilities in deep waters. They have been identified as one of the most cost-effective and promising concepts for future oil and gas development in the emerging frontiers. The anchor is released from a designed height above the seabed. This allows the anchor to gain velocity as it falls freely through the water column before impacting and embedding within the sediments.

The most commonly used DIAs are rocket-shaped, referred to as torpedo anchors, typically 12~17 m long and 0.8~1.2 m in diameter, with a dry weight (W_d) of 230~1150 kN, and may feature up to 4 fins at the trailing edge (see Figure 1; Brandão et al. 2006). They are released from a height of 50~100 m from the seabed, achieving velocities up to 35 m/s. Challenges associated with dynamically penetrating anchors include prediction of the anchor embedment depth and the subsequent capacity. The former is complicated by the very high strain rate (exceeding 25 s^{-1}) at the soil anchor interface, resulting from the high penetration velocities. There is general agreement that the undrained strength increases with increasing shear strain rate (e.g. Biscontin and Pestana 2001; DeGroot et al. 2007; Lunne and Andersen 2007; Dejong et al. 2012). Furthermore, natural soils also undergo softening as they are sheared and remoulded, with typical sensitivity values ranging from 2 to 5 for marine clays and 2 to 2.8 for reconstituted kaolin clay used widely in centrifuge tests (Kvalstad et al. 2001; Andersen and Jostad 2004; Randolph 2004; Menzies and Roper 2008; Lunne et al. 2011; Gaudin et al. 2014).

The paper is a continuation of one that presents results from a parametric study: exploring the relevant range of parameters in terms of anchor length; diameter; tip angle; number, width and length of fins; impact velocity and soil undrained shear strength (Kim et al. 2015). In that

paper, large deformation finite element (LDFE) analyses were carried out, accounting for the effect of strain rate and softening, but corresponding to a particular (kaolin) clay. In this study, the reverse was undertaken i.e. a typical torpedo anchor geometry and impact velocity were considered and parametric analyses were performed varying the soil sensitivity, brittleness and strain rate properties. The influence of these parameters on the proposed design expressions for anchor embedment depth was quantified. Analyses were also conducted simulating ideal Tresca, i.e. rate independent and non-softening, material for comparison.

Extensive background information to installation of torpedo anchors can be found in Hossain et al. (2014, 2015) and Kim et al. (2015), which are not repeated here. For convenience, Figures 1 and 2 from Kim et al. (2015) are used here, showing a typical anchor geometry defining the nomenclature adopted for the problem and typical mesh details respectively.

Numerical Analysis

Geometry and Parameters

This study has considered a torpedo anchor, consisting of a circular shaft attached with 4 rectangular fins, penetrating dynamically into a soft non-homogeneous clay deposit as illustrated schematically in Figure 1, where the mudline strength s_{um} , increases linearly with depth z , with a gradient k . The soil average effective unit weight is γ' . The anchor shaft diameter is D_A of 1.07 m, shaft length L_A of 17 m (including tip length, L_T), fin length L_F of 10 m ($= L_{F1} + L_{F2} + L_{F3}$) and fin width w_F of 0.9 m. Analyses were undertaken for anchors with a 30° conical tip ($\beta_{tip} = 30^\circ$). The shape was chosen similar to the T-98 anchor in the field, as illustrated by Medeiros (2002), de Araujo et al. (2004) and Brandão et al. (2006).

Analysis Details

3D LDFE analyses were carried out using the coupled Eulerian-Lagrangian (CEL) approach in the commercial package ABAQUS/Explicit (Dassault 2011). To reduce the computational effort, the anchor dynamic installation was modelled from the soil surface, with a given velocity v_i .

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 very fine soil mesh was necessary to capture the anchor-soil contact accurately. Based on the mesh convergence studies from Kim et al. (2015), the typical soil element size along the trajectory of the anchor is adopted as $0.019D_A$. 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.

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

The soil-anchor interface was modelled as frictional contact, using a general contact algorithm and specifying a Coulomb friction law together with a limiting shear stress ($\tau_{\max}$) along the anchor-soil interface. The Coulomb friction coefficient was deliberately set to a high value of $\mu_c = 50$, in order to allow the value of $\tau_{\max}$ 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 $S_{u,\text{ref}}$ at that depth; and (3) setting $\tau_{\max}$ equal to an interface friction ratio, α , times the calculated $S_{u,\text{ref}}$ 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, $\tau_{\max}$ is a constant value along the anchor surfaces during the entire calculation. At shallow depth, where $\tau_{\max}$ may exceed the rate dependent shear strength of the adjacent soil, failure may occur at the nearest integration points, rather than at the interface. The contact interface is created between Lagrangian mesh and Eulerian material, and automatically computed and tracked during the analysis.

Incorporation of Combined Effects of Strain Rate and Strain Softening

For this high velocity problem, a Herschel-Bulkley rheological model (H-B, Herschel and Bulkley 1926) is considered to be superior to the semi-logarithmic rate law for simulating geotechnical problems involving high strain rate (Raie and Tassoulas 2009; Zhu and Randolph 2011; Boukpeti et al. 2012). Except for the rate dependency, the softening induced by accumulated plastic strains is involved. The undrained shear strength at individual Gauss points was modified immediately after each step, according to the average rate of maximum shear strain in the previous time step and the current accumulated absolute plastic shear strain, as

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

where $s_{u,\text{ref}}$ is the shear strength at the reference shear strain rate of $\dot{\gamma}_{\text{ref}}$. The first bracketed term of Equation 1 augments the strength according to the operative shear strain rate, $\dot{\gamma}$, 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). 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. The parameter η is a viscous property and β the shear-thinning index. Boukpeti et al. (2012) carried out a series of undrained shear strength measurements on two different (kaolin and Burswood) clays from fall cone tests, vane shear, T-bar and ball penetrometer tests. They reported typical values of η and β in the range of $0.1 \sim 2.0$ and $0.05 \sim 0.15$, respectively, using $\dot{\gamma}_{\text{ref}} = 0.06 \text{ s}^{-1}$. O'Loughlin et al. (2013) and Chow et al. (2014) used $\dot{\gamma}_{\text{ref}} = 0.17 \text{ s}^{-1}$ and 0.1 s^{-1} for analysing centrifuge test data from installation of torpedo anchors and a dynamic cone penetrometer, respectively, in kaolin clay. Based on these, here a value of $\dot{\gamma}_{\text{ref}} = 0.1 \text{ s}^{-1}$ was adopted, and rate parameters were varied as $\eta = 0.1 \sim 1.5$ and $\beta = 0.05 \sim 0.2$. The shear strain rate, $\dot{\gamma}$, within the soil was evaluated according to

$$\dot{\gamma} = \frac{\Delta \epsilon_1 - \Delta \epsilon_3}{\Delta t} \quad (2)$$

where $\Delta \epsilon_1$ and $\Delta \epsilon_3$ are the cumulative major and minor principal strains, respectively, over the incremental time, Δt .

The second part of Equation 1 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 ~ 30 (i.e. 1,000~3,000% shear strain; Randolph 2004; Zhou and Randolph 2009).

The range of parameters considered is summarised in Table 1, encompassing most cases of practical interest. The base-case parameters were taken as $\dot{\gamma}_{ref} = 0.1 \text{ s}^{-1}$ and $v_i = 20 \text{ m/s}$.

Effects Evaluation and Discussion

Details of failure mechanisms and penetration trends associated with different torpedo anchor configurations have been documented by Kim et al. (2015). Results in the following sections will focus mainly on the effect of parameters pertinent to strain rate and strain softening leading to development of quantitative design charts in relation to the results for ideal Tresca material.

Effect on Failure Mechanisms

Figure 3a shows the anchor penetration depth, as a function of time, in a soil with $S_{u,ref} = 5 + 2z \text{ kPa}$ ($kD_A/S_{u,ref} = 0.43$, $S_{u,ref}/\gamma'D_A = 0.78$; Table 1). Three profiles correspond to (i) the ideal rate independent ($\eta = 0$), non-softening soil ($\delta_{rem} = 1$), (ii) rate dependent, strain softening soil ($\xi_{95} = 20$, $\eta = 1.5$, $\beta = 0.2$, $\dot{\gamma}_{ref} = 0.1 \text{ s}^{-1}$, $\delta_{rem} = 0.33$), and (iii) rate dependent, non-softening soil ($\xi_{95} = 20$, $\eta = 1.5$, $\beta = 0.2$, $\dot{\gamma}_{ref} = 0.1 \text{ s}^{-1}$, $\delta_{rem} = 1$). The soil failure mechanisms and velocity profiles corresponding to (i) and (ii) are shown in Figure 3b. Clearly,

the anchor penetration profile and corresponding soil failure mechanisms can be divided into two stages. Stage 1 corresponds to shallow penetration where the anchor accelerates although it advances into the soil. The soil resistance was less than the submerged weight of the anchor. The combined effect of various soil parameters in terms of rate dependency and strain softening (η , β , δ_{rem} , ξ_{95}) is minimal on the depth of stage 1. This is, at least partly, due to separation of the shaft and fins from the adjacent soils at the sharp transition zones between the anchor tip and shaft and shaft and fins (Figure 3c), and corresponding reduction in frictional resistances.

In Stage 2, as soil strength increases with depth, the anchor penetration resistance (frictional plus end bearing) overcome the submerged weight, negated by the buoyancy force, of the anchor and the anchor decelerates. This leads to a reduction of the extent of soil deformation at the tip and the base of the anchor shaft and fins (e.g. at $t = 0.9$ s; see Figure 3b). Finally, the soil movement nearly diminishes with the anchor achieving its final embedment depth i.e. coming to the rest. The penetration velocity for rate dependent and strength-softened soil decreases faster compared to that for ideal Tresca material, leading to an increasingly lower penetration depth. This is caused by the enhancement of the local shear strength (cross-coupling between the effects of strain rate and strain softening) and hence frictional and bearing resistances. The final embedment depth for the rate dependent, strain softening soil is lower than that for ideal Tresca material. This also confirms that rate dependency and softening have influence on deep penetration.

Effect on Embedment Depth

The embedment depth of a torpedo anchor on non-homogeneous clays is affected by a number of factors: (a) the strain rate parameters (η and β); (b) the strain softening parameter (ξ_{95}); (c) the remoulded ratio (δ_{rem}) and the soil sensitivity (S_t); (d) the reference strain rate

and ($\dot{\gamma}_{\text{ref}}$); (e) the impact velocity (v_i); and (f) the clay strength ($s_{u,\text{ref}}$). The results are discussed below in relation to these various factors. In all the embedment depth curves, the anchor penetration is plotted relative to the depth of the anchor tip below the original ground surface. The computational parameters were adopted in the CEL analyses (e.g. the soil element size along the trajectory of the anchor of $0.019D_A$) based on the results of the convergence studies reported by Kim et al. (2015).

Effect of Rate Parameters (η and β)

The rate parameters, η and β , were varied between 0 and 1.5 and 0.05 and 0.2 respectively. The effect on the embedment depth for upper ($\eta = 1.5$ and $\beta = 0.2$), intermediate ($\eta = 1.0$ and $\beta = 0.1$) and lower bound ($\eta = 0.1$ and $\beta = 0.05$) combinations of rate parameters are plotted in Figure 4 for ductility parameter of $\xi_{95} = 20$ (all the results will be discussed later). The result for ideal rate independent, non-softening soil is also included for comparison. Overall, the embedment depth reduces with increasing viscous property (η) and shear-thinning index (β), indicating the effect of enhancing undrained shear strength.

Interestingly, it is apparent that for high rate dependency, $\eta = 1.5$ and $\beta = 0.2$, the effect of strain softening is small and the curve lies above the one for ideal Tresca material. However, as the rate dependency becomes lower, the influence of softening becomes more marked, and the curve lies below the one for ideal Tresca material. For instance, for $v_i = 20$ m/s, $\xi_{95} = 20$ and $\delta_{\text{rem}} = 0.33$ (see Figure 4), the highest embedment depth $d_{e,t} = 33.06$ m was for $\eta = 0.1$ and $\beta = 0.05$, which is about 2.6% higher than that for ideal Tresca material. This gap reduces gradually and finally the lowest embedment depth of $d_{e,t} = 30.4$ m was for $\eta = 1.5$ and $\beta = 0.2$, which is about 5.6% lower than that for ideal Tresca material.

300 **Effect of Strain softening Parameter (ξ_{95})**

301 In order to show the effect of soil ductility, time-penetration profiles are shown in Figure 5 for
302 $\xi_{95} = 20$ and 30 ($v_i = 20$ m/s, $\eta = 1.0$, $\beta = 0.1$, $\delta_{rem} = 0.33$, $\dot{\gamma}_{ref} = 0.1$ /s, $kD_A/\text{Sum}_{ref} = 0.43$,
303 $\text{Sum}_{ref}/\gamma'D_A = 0.78$; Table 1). The result for non-softening soil is included in the figure for
304 comparison. It is seen that the influence of soil brittleness on embedment is minimal and
305 increases slightly with decreasing strain softening parameter, ξ_{95} (i.e. increasing brittleness).
306 For instance, for $v_i = 20$ m/s in Figure 5, the embedment depth $d_{e,t}$ increases from 31.88 to
307 32.16 m (i.e. 0.8%) as ξ_{95} reduces from 30 to 20.

308 **Effect of Soil Sensitivity (S_t)**

309 Typical sensitivity values for marine clays may range from 2 to 5 (Kvalstad et al. 2001;
310 Andersen and Jostad 2004; Randolph 2004; Menzies and Roper 2008; Lunne et al. 2011;
311 Gaudin et al. 2014). The effect of soil sensitivity, and hence remoulded strength ratio, δ_{rem} ,
312 was explored varying δ_{rem} from 0.2 to 0.5. The results are shown in Figure 6 with $\eta = 1.0$ and
313 1.5 , $\beta = 0.1$ and 0.2 , $\xi_{95} = 20$, $\dot{\gamma}_{ref} = 0.1$ s⁻¹ ($kD_A/\text{Sum}_{ref} = 0.43$ and $\text{Sum}_{ref}/\gamma'D_A = 0.78$; Table 1).
314 As expected, the embedment depth increases with increasing soil sensitivity (S_t). For instance,
315 for $\eta = 1.0$ and $\beta = 0.1$, the highest embedment depth, $d_{e,t} = 32.39$ m for $S_t = 5$, which is about
316 2 % higher than that for $S_t = 2$. Similar difference can be quantified for higher rate parameters
317 ($\eta = 1.5$, $\beta = 0.2$).

318 **Combined Effects of Strain Rate, Strain Softening, and Soil Sensitivity**

319 In order to develop a design chart for estimating the embedment depth, the relationships
320 between strain rate and strength-softening were identified from all the anchor penetration
321 analyses tabulated in Table 1. It was found that the final embedment depth of anchor was

approximately inverse proportional to the strain rate parameters (η and β) and the strain softening parameters (δ_{rem} and ξ_{95}).

From the parametric study encompassing a range of relevant values of $\eta = 0\sim 1.5$, $\beta = 0.05\sim 0.2$, $\xi_{95} = 20\sim 30$, and $\delta_{rem} = 0.2\sim 0.5$ (see Table 1), it is attempted to develop an approximate expression for anchor embedment depth, $d_{e,t}$, accounting for the combined effects of strain rate and strain softening. The results for soil with $s_{u,ref} = 5 + 2z$ kPa ($kD_A/s_{u,ref} = 0.43$, $s_{u,ref}/\gamma'D_A = 0.78$) are presented in Figures 7a~7d with the normalised embedment depth, $d_{e,t}/D_p$, plotted against viscosity property η for different β and ξ_{95} . Values for strain-softened clay follow the similar trend, but lie well below the curve for non-softening clay, reflecting the reduction in average strength ratio within the failure mechanism, which increases final anchor embedment.

A convenient expression for estimating the normalised embedment depth, $d_{e,t}/D_p$, in rate dependent and strain-softened clay is expressed as

$$\frac{d_{e,t}}{D_p} = \frac{1 + \eta}{[1 + \eta(R_a)^\beta][\delta_{rem} + (1 - \delta_{rem})e^{-3\xi_a/\xi_{95}}]} \left(\frac{d_{e,t}}{D_p} \right)_{ideal} \quad (3)$$

where $(d_{e,t}/D_p)_{ideal}$ is the value of $d_{e,t}/D_p$ obtained for an ideal rate independent, non-softening Tresca material. This expression is calibrated with results from LDFE analyses in Figure 7 for $\delta_{rem} = 0.33$ (a sensitivity of 3) and $v_i = 20$ m/s. Although there is some cross-coupling between the effects of strain rate and strain softening, the multiplicative factors given in Equation 3 appear sufficiently accurate taking $R_a = 2.0$ and $\xi_a = 0.35$. The value of ξ_a represents an average cumulative shear strain experienced by soil particles as they traverse the flow mechanism around the anchor. The value of rate coefficient R_a reflects the higher average shear strain rate in the flow mechanism compared with the reference value. As shown

in Figure 7b, net influences of η and β for different ξ_{95} are almost identical, that is, all values are in similar proportion to the curve for non-softening but rate dependent soil.

Effect of Reference Strain Rate ($\dot{\gamma}_{\text{ref}}$)

In LDFE analysis results presented so far, the soil strength $s_{u,\text{ref}} = 5 + 2z$ kPa ($kD_A/s_{u,\text{ref}} = 0.43$, $s_{u,\text{ref}}/\gamma'D_A = 0.78$), impact velocity ($v_i = 20$ m/s) and reference shear strain rate ($\dot{\gamma}_{\text{ref}} = 0.1$ s⁻¹) were all kept constant. The rate coefficient, R_a , of 2.0 is applicable to the normalised velocity $v_i/D_A \dot{\gamma}_{\text{ref}} = 186.92$. In the field, torpedo anchor normalised penetration rate, v_i/D_A , may vary between 13 and 44 ($v_i = 15\sim 35$ m/s, $D_A = 0.8\sim 1.2$ m). In addition, reference shear strain rates, at which $s_{u,\text{ref}}$ was measured, may be different, with rates for laboratory triaxial test (~ 0.00001 s⁻¹), T-bar and ball penetrometer tests ($0.001 \sim 0.1$ s⁻¹), vane shear and viscometer tests ($0.01 \sim 100$ s⁻¹) and fall cone test ($1 \sim 10$ s⁻¹). An equivalent embedment depth for other normalised velocities may be obtained by adjusting the rate dependent term accordingly by the ratio of normalised velocities. Such adjustment would lead to negligible rate effect for $v_i/D_A \dot{\gamma}_{\text{ref}} = 10\sim 20$. In general though, anchor embedment depth would increase with increasing $\dot{\gamma}_{\text{ref}}$.

In order to investigate the effect of $v_i/D_A \dot{\gamma}_{\text{ref}}$ on the embedment depth, analyses were carried out varying $\dot{\gamma}_{\text{ref}}$ from 0.01 s⁻¹ (average of penetrometer tests) to 1 s⁻¹ (average of vane shear test and lower bound of fall cone test), but maintaining $v_i = 20$ m/s and $D_A = 1.07$ m (Table 1). The results are plotted in Figure 8 together with the curve for $v_i/D_A \dot{\gamma}_{\text{ref}} = 186.9$. And indeed, the embedment depth is $\sim 5\%$ lower for $v_i/D_A \dot{\gamma}_{\text{ref}} = 1869.16$ ($\dot{\gamma}_{\text{ref}} = 0.01$ s⁻¹) and $\sim 5\%$ higher for $v_i/D_A \dot{\gamma}_{\text{ref}} = 18.69$ ($\dot{\gamma}_{\text{ref}} = 1$ s⁻¹) compared to that for $v_i/D_A \dot{\gamma}_{\text{ref}} = 186.9$ ($\dot{\gamma}_{\text{ref}} = 0.1$ s⁻¹).

Reference shear strain rate $\dot{\gamma}_{\text{ref}}$ is directly linked to $s_{u,\text{ref}}$. Parametric analyses are carried out isolating parameters. The effect of $\dot{\gamma}_{\text{ref}}$ is highlighted in this section, with the effect of reference shear strength (and v_i) discussed in the following sections. This systematic investigation is consistent with the ones undertaken by e.g. Hossain and Randolph (2009) for spudcan foundations.

Effect of Impact Velocity (v_i)

Additionally, to show the effect of impact velocity (v_i), time-penetration profiles are presented in Figures 9a and 9b for varying v_i (15 ~ 30 m/s) and rate parameters, but keeping other parameters as constant ($\xi_{95} = 20$, $\delta_{\text{rem}} = 0.33$, $\dot{\gamma}_{\text{ref}} = 0.1 \text{ s}^{-1}$, $kD_A/s_{u,\text{ref}} = 0.43$, $s_{u,\text{ref}}/\gamma'D_A = 0.78$; Table 1). From Figure 9, the net increase in velocity at shallow embedment (Stage 1) is seen to reduce as the impact velocity increases. This is to be expected as the closer the impact velocities approach the anchor's terminal velocity, the less acceleration will occur within the soil. The anchor penetration depth at which the anchor begins to decelerate is also seen to be reduced with increasing impact velocity (see Figure 9b). A consistent trend was reported by O'Loughlin et al. (2013) analysing centrifuge model test data.

Overall, the embedment depth increases significantly with increasing impact velocity. Interestingly, the influencing ratio of strain rate and softening on the embedment depth is similar despite different impact velocities. For instance, for $v_i = 15 \text{ m/s}$ in Figure 9, the highest embedment depth $d_{e,t} = 30.01 \text{ m}$ was for $\eta = 0.1$, $\beta = 0.05$, $\xi_{95} = 20$ and $\delta_{\text{rem}} = 0.33$ which is about 8.1 % higher than that for $\eta = 1.5$, $\beta = 0.15$ and $\delta_{\text{rem}} = 1$. This gap remains somewhat constant (~8.0%) as the impact velocity increases from 20 m/s to 30 m/s.

Effect of Reference Shear Strength $s_{u,ref}$ or Soil Strength Non-homogeneity ($kD_A/s_{u,ref}$) and Normalised Strength ($s_{u,ref}/\gamma'D_A$)

Initial use of torpedo anchors took place at Marlim Field, Albacora Leste Field and Corvina Field in the Campos Basin (Medeiros 2002; Argiolas and Rosas 2003; de Araujo et al. 2004; Brandão et al. 2006). A similar concept has been used at the Vøring Plateau, Troll Field and Gjøa Field in the North Sea (Lieng et al. 2010; Sturm et al. 2011) and in the Gulf of Mexico (Wodehouse et al. 2007; Zimmerman et al. 2009). Hossain et al. (2014) provided a summary of these field tests. In these regions, seabed sediments are mostly composed of normally consolidated clay, with strength increasing somewhat linearly with depth. Typical strengths at the seabed level of $s_{um} = 2\sim 15$ kPa, and gradients, k , in the range of $0\sim 3$ kPa/m. The effective unit weight of soil, γ' , typically varies between 5 and 9 kN/m³. In order to examine the corresponding effect on the embedment depth in rate dependent and strain softening clay, analyses were also undertaken for $s_{u,ref} = 1 + 1z$ kPa ($kD_A/s_{u,ref} = 1.07$, $s_{u,ref}/\gamma'D_A = 0.16$) and $s_{u,ref} = 10 + 3z$ kPa ($kD_A/s_{u,ref} = 0.32$, $s_{u,ref}/\gamma'D_A = 1.56$), and the results are shown in Figure 10. The curve for $s_{u,ref} = 5 + 2z$ kPa ($kD_A/s_{u,ref} = 0.43$, $s_{u,ref}/\gamma'D_A = 0.78$) is also included in Figure 10 for comparison. The curve for lower shear strength (lower non-homogeneity, but higher normalised strength) attain to a plateau earlier, resulting in a lower embedment depth. It is seen that effect of $s_{u,ref}$ and $kD_A/s_{u,ref}$, $s_{u,ref}/\gamma'D_A$ is markedly profound relative to other effects as the soil undrained shear strength is the dominant factor for anchor penetration resistance in stage 2 (see later Equation 4). Compared to that ($d_{e,t} = 32.16$ m) for $s_{u,ref} = 5 + 2z$ kPa ($kD_A/s_{u,ref} = 0.43$, $s_{u,ref}/\gamma'D_A = 0.78$), the embedment depth for $s_{u,ref} = 1 + 1z$ kPa ($kD_A/s_{u,ref} = 1.07$, $s_{u,ref}/\gamma'D_A = 0.16$) is 35% higher and for $s_{u,ref} = 10 + 3z$ kPa ($kD_A/s_{u,ref} = 0.32$, $s_{u,ref}/\gamma'D_A = 1.56$) is 16% lower.

Prediction of Embedment Depth in Field

Extended Bearing Resistance Method

Validation was undertaken against field data on a 4-fin torpedo anchor, as reported by Brandão et al. (2006). Eighteen torpedo anchors ($W_s = 850$ 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; $kD_A/s_{u,ref} = 0.43$ and $s_{u,ref}/\gamma'D_A = 0.78$) at Campos basin, Brazil. The achieved impact velocity was $v_i \approx 26.8$ m/s and the average penetrations were about 35.2 m. The objective in this part is to compare predicted penetrating depth profile obtained using a bearing resistance method.

The motion response of a torpedo anchor during dynamic embedment in soil may be approached by considering Newton's second law of motion and the forces acting on the anchor during penetration. 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, 2015; Kim et al. 2015) have adopted such an approach, with variations on the inclusion and formulation of the various forces acting on a torpedo anchor. A similar approach is adopted here

$$\begin{aligned} m \frac{d^2z}{dt^2} &= W_s - F_\gamma - R_{f1}F_b - R_{f2}F_f - F_d \\ &= W_s - F_\gamma - R_{f1}(F_{b,bA} + F_{b,bF}) - R_{f2}(F_{fA} + F_{fF}) - F_d \\ &= W_s - F_\gamma - R_{f1}(N_{c,bA}s_{u,bA}A_A + N_{c,bF}s_{u,bF}A_{bF}) - \alpha R_{f2}(s_{u,sA}A_{sA} + s_{u,sF}A_{sF}) \\ &\quad - \frac{1}{2}C_d\rho_s A_p v^2 \end{aligned} \quad (4)$$

The terms used in the above expression are defined under nomenclature. R_{f1} and R_{f2} are shear strain rate and softening related terms 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. W_s is the submerged weight (in water) of the anchor and F_γ is a buoyancy force, calculated as the displaced volume times the effective unit weight of the soil (γ'). 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 velocity of the penetrating anchor). During the penetration process, soil is displaced from the path of the advancing anchor. Soil elements are accelerated from rest to a velocity sufficient to move them out of the path of the anchor. The force required to accelerate the soil elements is known as an inertial force. The reaction to the inertial force produces a resistance force on the anchor. Inertial drag resistance in soil is analogous to the hydrodynamic drag experienced by an object passing through water. Considering the very soft, viscous clay typically present at the seabed surface it seems reasonable to assume that an inertial drag force exists during penetration (Richardson et al. 2009; Hasanloo et al. 2012).

The dependence of shear strength on shear strain rate and strain softening can be generally formulated using Equation 5 and has been accounted for in Equation 4 by scaling the bearing resistances by a rate function, R_{f1} , expressed as

$$R_{f1} = \frac{1}{(1+\eta)} \left[1 + \eta \left(\frac{nv/D_A}{\dot{\gamma}_{ref}} \right)^\beta \right] \left[\delta_{rem} + (1 - \delta_{rem}) e^{-3\xi_a/\xi_{95}} \right] \quad (5)$$

In this analysis, values for the various parameters were adopted as follows: $C_d = 0.63$ (Liang et al. 1999, 2000, 2010; Sturm et al. 2011; O’Loughlin et al. 2013), $n = 1.0$, $N_{c,bA} = 13.56$, $N_{c,bF} = 7.5$, $\eta = 1.0$, $\beta = 0.1$, $\delta_{rem} = 1/S_t = 1/3$, and $\xi_{95} = 20$. For the frictional resistance, R_{f2} was taken as $2R_{f1}$ from the previous reports (Einav and Randolph 2006; Chow et al. 2014;

Steiner et al. 2014). The predictions made the same assumptions based on the findings from the parametric studies, adopting $\dot{\gamma}_{\text{ref}} = 0.1 \text{ s}^{-1}$ and $\xi_a = 0.35$.

Figure 11a shows the measured (Brandão et al. 2006), computed (LDFE analysis) and predicted velocity-penetration profiles using Equations 4 and 5 ($kD_A/\text{Sum}_{\text{ref}} = 0.43$, $\text{Sum}_{\text{ref}}/\gamma'D_A = 0.78$). The estimated profiles assuming ideal rate independent, non-softening ($\eta = 0$, $\delta_{\text{rem}} = 1$; Equation 5) and rate dependent, non-softening ($\delta_{\text{rem}} = 1$; Equation 5) soil are also included in the figure. Overall, the computed and predicted curves for rate dependent and softening soil closely predict the general trend of the measured data, and the predicted embedment depth ($d_{e,t}$) also shows a good agreement with an error of $< \pm 4.0\%$. Lower and significantly higher embedment depths were resulted for rate dependent, non-softening and rate independent, non-softening soil respectively, confirming the necessity of considering in particular rate dependency for this dynamic installation problem. The motion of the anchor and hence the magnitude of the penetration resistance forces may be estimated from Equations 4 and 5. The contributions of these resistance forces are plotted in Figure 11b. The figure confirms that the total resistance force at relatively shallow depths (Stage 1, acceleration stage) does not exceed the weight of the anchor.

Extended Modified Energy Method

Recently, O'Loughlin et al. (2013) proposed a simple expression utilising a dataset of centrifuge testing. Total energy (E_{total}), defined as the sum of the kinetic energy of the anchor at the mudline and the potential energy released as it penetrates in the seabed, normalised by k and D_p^4 (i.e. E_{total}/kD_p^4) were used to link to the normalised embedment depth ($d_{e,t}/D_p$). In an attempt to improve the total energy method, Kim et al. (2015) carried out a series of LDFE analyses for various anchor configurations (but with a fixed series of strain rate and softening parameters: $\dot{\gamma}_{\text{ref}} = 0.1 \text{ s}^{-1}$, $\eta = 1.0$, $\beta = 0.1$, $\delta_{\text{rem}} = 1/S_t = 1/3$, $\xi_{95} = 20$) and modified it as a

478 function of $E_{\text{total}}/kA_s D_p^2$ accounting for the contribution of the total surface area of the anchor
 479 ($A_s = A_{sA} + A_{sF}$) as

$$480 \quad \frac{d_{e,t}}{D_p} = 4.0 \left(\frac{E_{\text{total}}}{kA_s D_p^2} \right)^{0.389} \quad (6)$$

481 where

$$482 \quad E_{\text{total}} = \frac{1}{2} m v_i^2 + m' g d_{e,t} \quad (7)$$

483 with m' being the effective mass of the anchor (submerged in soil), g Earth's gravitational
 484 acceleration of 9.81 m/s^2 and D_p the anchor projected area equivalent diameter (including
 485 fins). Figure 12 shows the relationship following Equation 6, and data for various velocities
 486 and strain rate and strain softening parameters. For each combination of β and δ_{rem} (or non-
 487 softening), four analyses were carried out varying η as 0, 0.5, 1.0 and 1.5, as labelled in
 488 Figure 12. By comparing the result for the set parameter ($\eta = 1.0$, $\beta = 0.1$, $\delta_{\text{rem}} = 1/St =$
 489 $1/3$, $\xi_{95} = 20$), anchor embedment depth decreases or increases depending on increasing or
 490 decreasing of the degree of rate dependency and the inverse of the sensitivity of the soil. From
 491 the parametric study encompassing a range of relevant values of $\eta = 0.1 \sim 1.5$; $\beta = 0.05 \sim$
 492 0.2 ; $\delta_{\text{rem}} = 0.2 \sim 0.5$; and $\xi_{95} = 20 \sim 30$ (see Table 1), Equation 6, accounting for the
 493 combined effects of strain rate and strain softening, may be re-written with a multiplicative
 494 term as

$$495 \quad \frac{d_{e,t}}{D_p} = \frac{1 + \eta}{[1 + \eta(R_b)^\beta][\delta_{\text{rem}} + (1 - \delta_{\text{rem}})e^{-3\xi_b/\xi_{95}}]} 4.0 \left(\frac{E_{\text{total}}}{kA_s D_p^2} \right)^{0.389} \quad (8)$$

496 By comparing this expression with modified energy method, as shown in Figure 12, values
 497 for the parameters R_b and ξ_b may be estimated as $R_b \sim 1.8$ and $\xi_b \sim 0.28$. The rate coefficient

and average cumulative shear strain in Figure 7 are slightly higher ($R_a = 2.0$, $\xi_a = 0.35$), since the last term of Equation 3 obtained for an ideal rate independent, non-softening clay. Figure 12 confirms, as shown by two bound lines, that the modified method with multiplication term (Equation 8) can predict anchor embedment depths accurately for various range of strain rate dependency, strain softening, soil strength and sensitivity, and anchor impact velocity.

Concluding Remarks

Dynamic installation of a typical torpedo anchor was investigated extensively through 3D dynamic large deformation finite element analyses. The embedment depth of the anchor was found to be a function of the impact velocity, strain rate dependency of the shear strength, degree of remoulding during shearing (or strain softening) and hence sensitivity of the soil. The effect of the geometric parameters of torpedo anchors such as the length and diameter of the shaft; number, width and length of fins; and the undrained shear strength of seabed sediments were reported by Kim et al. (2015).

Overall, the effect of strain rate dependency on the anchor penetration profile was found to dominate. Although the effects of rate dependency were partly compensated for by an increase in penetration depth resistance due to strain softening, the strain rates involved were very high (i.e. the softening was relatively quite small), resulting in correspondingly small strain softening effects. During penetration of the anchor, the mobilised soil strength was affected by strain rate and accumulated strain softening simultaneously. The soil with larger strain softening (lower strength) was also featured with higher strain rate, which tended to increase the soil strength, i.e. the effects of strain softening and strain rate in ‘shear bands’ are partly compensated each other.

A robust total energy approach, modified from that proposed by O’Loughlin et al. (2013), was proposed (Equation 8), taking into account the effect of anchor mass, impact velocity, surface area, projected area equivalent diameter of the anchor, the soil undrained shear strength, strain rate dependency of the shear strength, degree of remoulding during shearing (or strain softening) and hence sensitivity of the soil.

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Table 1. Summary of Selected Parameters for LDFE Analysis

$kD_A/\text{Sum}_{\text{ref}}$	$\text{Sum}_{\text{ref}}/\dot{\gamma}'D_A$	$v_i/D_A\dot{\gamma}_{\text{ref}}$	η	β	ξ_{95}	δ_{rem} (1/St)	Notes
0.43	0.78	186.91	0, 0.1, 0.5, 1.0, and 1.5	0.05, 0.10, 0.15 and 0.20	20	0.33	Effect of η and β
0.43	0.78	186.91	0, 0.1, 0.5, 1.0, and 1.5	0.05, 0.10, 0.15 and 0.20	-	-	Without softening
0.43	0.78	186.91	0.5, 1.0 and 1.5	0.10	20 and 30	0.33	Effect of ξ_{95}
0.43	0.78	186.91	1.0 and 1.5	0.10 and 0.20	20	0.20, 0.33 and 0.50 (1/5, 1/3, 1/2)	Effect of δ_{rem}
0.43	0.78	18.69 and 1869	1.0	0.10	20	0.33	Effect of $\dot{\gamma}_{\text{ref}}$
0.43	0.78	-	-	-	-	-	Ideal Tresca material
1.07	0.16	186.91	1.0	0.10	20	0.33	Effect of $kD_A/\text{Sum}_{\text{ref}}$
0.32	1.56						

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645 **No. of Figure: 12**

646 **Figure 1.** Schematic diagram of installed torpedo anchor in clay

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

649 **Figure 3.** Effect of strain rate and strain softening on embedment depths and soil failure
650 mechanisms ($L_A = 17$ m, $D_A = 1.07$ m, $L_F = 10$ m, $w_F = 0.9$): (a) Embedment
651 depths; (b) Instantaneous (resultant) velocity vectors with anchor embedment;
652 (c) Soil separation at shallow depth

653 **Figure 4.** Effect of strain softening parameters η and β on anchor embedment

654 **Figure 5.** Effect of strain softening parameter ξ_{95} on anchor embedment depth

655 **Figure 6.** Effect of remoulded strength ratio δ_{rem} and soil sensitivity S_t on anchor
656 embedment depth

657 **Figure 7.** Combined effect of strain rate parameter η , β and strain softening parameter
658 ξ_{95} on anchor embedment depth: (a) $\beta = 0.05$; (b) $\beta = 0.1$; (c) $\beta = 0.15$; (d) $\beta =$
659 0.2

660 **Figure 8.** Effect of reference shear strain $\dot{\gamma}_{ref}$ on anchor embedment depth

661 **Figure 9.** Effect of impact velocity v_i on anchor embedment depth: (a) Time-embedment
662 profile; (b) Velocity profile

663 **Figure 10.** Effect of soil undrained shear strength on anchor embedment depth

664 **Figure 11.** Measured vs estimated anchor installation profile using proposed bearing
665 resistance method: (a) Comparison between measured field installation data
666 and estimated profile; (b) Contribution of various resistance force components
667 (Equation 4: rate dependent & strain softening)

668 **Figure 12.** Modified energy method for assessing anchor embedment depths: (a)
669 Estimation using modified energy method; (b) Magnification of numerical
670 dataset
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Figure 1. Schematic diagram of installed torpedo anchor in clay

(a) Typical 3D mesh

(b) Detail plan view

(c) Side view

Figure 2. Typical mesh used in CEL analysis

3(a) Embedment depths

3(b) Instantaneous (resultant) velocity vectors with anchor embedment

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3(c) Soil separation at shallow depth

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Figure 3. Effect of strain rate and strain softening on embedment depths and soil failure mechanisms ($L_A = 17 \text{ m}$, $D_A = 1.07 \text{ m}$, $L_F = 10 \text{ m}$, $w_F = 0.9$)

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698 **Figure 4. Effect of strain softening parameters η and β on anchor embedment**

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Figure 5. Effect of strain softening parameter ξ_{95} on anchor embedment depth

Figure 6. Effect of remoulded strength ratio δ_{rem} and soil sensitivity S_t on anchor embedment depth

7(a) $\beta = 0.05$

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7(b) $\beta = 0.1$

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7(c) $\beta = 0.15$

Figure 7. Combined effect of strain rate parameter η , β and strain softening parameter ξ_{95} on anchor embedment depth

Figure 8. Effect of reference shear strain rate $\dot{\gamma}_{ref}$ on anchor embedment depth

9(a) Time-penetration profile

9(b) Velocity-penetration profile

Figure 9. Effect of impact velocity v_i on anchor embedment depth

Figure 10. Effect of soil undrained shear strength on anchor embedment depth

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11(a) Comparison between measured field installation data and estimated profile

11(b) Contribution of various resistance force components (Equation 4: rate dependent & strain softening)

Figure 11. Measured vs estimated anchor installation profile using proposed bearing resistance method

12(a) Estimation using modified energy method

12(b) Magnification of numerical dataset

Figure 12. Modified energy method for assessing anchor embedment depths