Assessing the penetration resistance acting on a dynamically installed anchor in normally and over consolidated clay

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| Complete List of Authors: | O'Beirne, Colm; University of Western Australia, Centre for Offshore Foundation Systems  
                      O'Loughlin, Conleth; University of Western Australia, Gaudin, Christophe; University of Western Australia, Centre for Offshore Foundations Systems  |
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Assessing the penetration resistance acting on a dynamically installed anchor in normally and over consolidated clay

C. O’Beirne*

Centre for Offshore Foundation Systems, Perth, Australia

C.D. O’Loughlin

Centre for Offshore Foundation Systems, Perth, Australia

C.Gaudin

Centre for Offshore Foundation Systems, Perth, Australia

*Corresponding Author: colm.obeirne@research.uwa.edu.au
Abstract
Predicting the final embedment depth of a dynamically installed anchor is a key prerequisite for reliable calculation of anchor capacity. This paper investigates the embedment characteristics of dynamically installed anchors in normally and over consolidated clay through a series of centrifuge tests involving a model anchor instrumented with a MEMS accelerometer, enabling the full motion response of the anchor to be established. The data are used to assess the performance of an anchor embedment model based on strain-rate-dependent shearing resistance and fluid mechanics drag resistance. Predictions of a database of over 100 anchor installations – formed from this study and the literature – result in calculated anchor embedment depths that are within ±15% of the measurements. An interesting aspect, consistent across the entire database, relates to the strain rate dependence on frictional resistance relative to bearing resistance. The predictions reveal that strain rate dependency may indeed be higher for frictional resistance, although only if a soil strength lower than the fully remoulded strength is considered as the reference strength, which suggests that water may be entrained along a boundary layer at the anchor-soil interface during installation.

Keywords: dynamically installed anchor, MEMS, soft soil, centrifuge modelling
Notation

\( a_{\text{linear}} = \) linear acceleration
\( a, b, c, d, e, f = \) fitting constants
\( A_p = \) anchor projected area
\( A_s = \) anchor frictional area
\( C_D = \) drag coefficient
\( c_h = \) coefficient of horizontal consolidation
\( D = \) anchor shaft diameter
\( d = \) T-bar diameter
\( F_b = \) buoyancy force
\( F_{\text{bear}} = \) bearing resistance
\( F_d = \) drag resistance
\( F_{\text{frict}} = \) frictional resistance
\( F_{\text{net}} = \) net force
\( k = \) undrained shear strength gradient with depth
\( L = \) anchor length
\( m = \) mass
\( m' = \) added mass
\( N = \) acceleration level
\( n, n_1 = \) strain rate parameters
\( N_c = \) bearing capacity factor
\( r = \) radius
\( R_f = \) strain rate function
\( R_{f,\text{bear}}, R_{f,\text{frict}} = \) strain rate functions for bearing and frictional resistance
\( s_u = \) undrained shear strength
\( s_{um} = \) undrained shear strength at mudline
\( t = \) time
\( v = \) velocity
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<tr>
<td>$W$</td>
<td>anchor dry weight</td>
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<tr>
<td>$z$</td>
<td>depth, anchor displacement</td>
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<tr>
<td>$z_{\text{deep}}$</td>
<td>depth at which hole-closure occurs</td>
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<td>$z_{e,\text{tip}}$</td>
<td>anchor tip embedment</td>
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<td>$\alpha$</td>
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<tr>
<td>$\gamma'$</td>
<td>effective unit weight</td>
</tr>
<tr>
<td>$\Delta z_{\text{scrape}}$</td>
<td>depth of soil scraped from sample surface</td>
</tr>
<tr>
<td>$\Lambda$</td>
<td>plastic volumetric strain ratio</td>
</tr>
<tr>
<td>$\rho_s$</td>
<td>soil density</td>
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<td>$\sigma'_v$</td>
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<td>$\dot{\gamma}$</td>
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<tr>
<td>$\dot{\gamma}_{\text{ref}}$</td>
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<tr>
<td>$\mu$</td>
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Introduction

The shift of oil and gas operations towards more remote locations, often in deeper water, tends to increase the costs associated with installing the anchors and laying the moorings. As much of this cost is due to the day-rate of installation vessels, there is an incentive to reduce the overall anchor installation durations, particularly for mobile drilling operations. This may be achieved by using anchors that are quick to install, such as dynamically installed anchors that free-fall through the water column and self-bury into the seabed (Figure 1). Various geometries have been proposed, although they tend to be rocket or torpedo shaped with overall lengths between 9 and 22 m, dry weights in the range 25 to 120 t and featuring three or four flukes (also referred to as fins) at the trailing edge.

Anchor capacity is derived from a combination of frictional resistance along the anchor length, bearing resistance on the anchor flukes and the submerged weight of the anchor. Evidently the final anchor embedment depth becomes a key design parameter as this will dictate the soil strength in the vicinity of the embedded anchor and in turn the anchor capacity. Predicting this final embedment depth is challenging due to the very high penetration velocities (up to ~30 m/s) that: (i) significantly increase the mobilised soil strength during dynamic embedment, and (ii) introduce hydrodynamic aspects, including potential water entrainment at the anchor-soil interface and pressure drag that occurs both during free-fall in water and during dynamic embedment in soil (O’Loughlin et al. 2014a).

Theoretical approaches for predicting the final anchor embedment depth have been proposed and their merit has been examined using data, mainly from centrifuge studies (e.g. O’Loughlin et al. 2004a, 2009, 2013; Richardson et al. 2006; Gaudin et al. 2013) in normally consolidated clay. However, limitations on the measurements available during dynamic penetration have limited the calibration and validation of these embedment models to known starting and end conditions: the measured anchor impact velocity (at zero embedment) and the measured anchor embedment (at zero anchor velocity).

Although this has been an important ‘first step’ in demonstrating the appropriateness of such models in predicting final embedment, an equally important ‘second step’ is to rigorously appraise these models using motion data measured during the dynamic embedment event. The complexity of the mechanisms that occur during dynamic embedment warrants this development, offering a basis for assessing parameter selection and the inclusion or exclusion of the various resistance terms. This
paper adopts such an approach, extending previous work by considering the motion response during penetration, but also by considering both normally and overconsolidated clay.

**Experimental details**

The motion data for further validating the anchor embedment model was obtained in centrifuge tests using a dynamically installed anchor instrumented with a micro-electro mechanical system (MEMS) accelerometer. The tests were conducted in both normally and overconsolidated clay so that the model could be applied to a range of overconsolidation ratios and strength profiles. Relevant experimental details are provided in the following sections.

**Soil samples**

Clay samples were prepared by consolidating kaolin slurry, with an initial moisture content of twice the liquid limit (120%), under self-weight in a beam centrifuge at an acceleration of 100 g for six days. During this time additional slurry was added to the sample surfaces to maintain a target sample height of ~ 220 mm. A 30 mm water layer was maintained at the sample surface to ensure sample saturation during consolidation and testing. The tests were conducted in three sample boxes, each with internal dimensions 598 mm long, 117 mm wide and 300 mm deep, nested side by side in a larger strongbox. A geotextile drainage blanket was placed along the base of the sample boxes and along the end walls to ensure two-way drainage during consolidation and that there was no hydraulic gradient between the top and base of the samples. Variations in the strength profile of the clay samples were achieved by periodically scraping a predetermined depth of clay from the sample surface and allowing a swelling period of approximately 15 hours before further testing. This had the effect of increasing both the strength of the clay at the mudline and the overconsolidation ratio (OCR).

**Instrumented model anchor**

The centrifuge tests were carried out at the consolidation acceleration level of 100 g using the model anchor shown in Figure 2. The model anchor was fabricated from aluminium and brass, and was based on an idealised geometry proposed by Lieng et al. (1999), with an elliptical tip and four large
flukes at the trailing end. At model scale the anchor has a length, $L = 75$ mm, a shaft diameter, $D = 6$ mm, and a mass, $m = 10.17$ g. At prototype scale the anchor length and mass is $7.5$ m and $10.17$ tonnes respectively, representing a half scale full size anchor. The model scaling was based on constraints on the maximum size that could be accommodated in the centrifuge and the maximum measurement range of the accelerometer used to measure the motion response during dynamic embedment.

The model anchor was instrumented with a single axis ± 500 g MEMS accelerometer (Analog Devices ADXL001) measuring $5 \times 5 \times 2$ mm, that was embedded within an epoxy resin filled void in the anchor shaft (as shown in Figure 2). The MEMS accelerometer can measure both constant and changing acceleration and was used to capture the motion response of the anchor during free-fall and embedment in soil. The three sensor wires (power, ground and signal) were recessed into the anchor shaft and exited at the rear of the shaft where they joined the anchor retrieval line.

**Experimental arrangement and procedures**

The experimental arrangement is shown in Figure 3; as it is similar to that described in detail by O’Loughlin et al. (2004a), only a brief description is provided here. The anchor was dynamically installed by allowing it to fall from a preselected height above the sample surface (typically $300$ mm) through a vertical installation guide (which applied the tangential force required to keep the anchor rotating at the same angular velocity as the centrifuge). Anchor release was achieved in-flight by supplying current to a resistor that caused it to heat and burn through a sacrificial release cord. Anchor velocity was primarily derived from the MEMS accelerometer measurements, although independent checks on the accelerometer-derived anchor velocity before impact with the soil was achieved using photo-emitter-receiver pairs (PERPs; see Richardson et al. 2006, Chow et al. 2014) that were located on the lower end of the installation guide (i.e. close to the sample surface). The PERPs also served as a pre-trigger for the accelerometer data that was captured at $50$ kHz using the high-speed logging mode in the data acquisition hardware (Gaudin et al. 2009) when the anchor passed the first PERP. After dynamic embedment the centrifuge was spun down and a direct measurement of anchor embedment was taken by clamping the retrieval line from the rear of the anchor at the sample surface,
extracting the anchor, and then measuring the vertical distance from the clamp to the anchor tip using a scale rule marked in half millimetre divisions.

Results and discussion

Undrained shear strength

Profiles of undrained shear strength, $s_u$, were determined for each sample from T-bar penetrometer tests, involving a penetration velocity, $v = 1$ mm/s, such that over the penetration depth the non-dimensional velocity $v_d/c_h = 10$ to 50 ($d$ is the T-bar diameter = 5 mm and $c_h$ is the coefficient of horizontal consolidation $\approx 0.1$ to 0.5 mm$^2$/s (Colreavy et al. 2016) and the response is primarily undrained (House et al. 2001). Typical $s_u$ profiles are shown in Figure 4, where the depth axis reflects the changing mudline surface with each scrape, represented by the sum of the current sample depth, $z$, plus the scrape depth, $\Delta z_{\text{scrape}}$. Also shown on Figure 4 are profiles of OCR with depth, which (with the exception of the normally consolidated sample, for which OCR = 1) reduce non-linearly with depth from OCR = 3.5 to 10.2 at 6 mm below the mudline (one anchor shaft diameter, $D$) to OCR = 1.1 to 1.6 at $z = 150$ mm. The profiles were interpreted from the measured penetration resistance according to the framework proposed by White et al. (2010), which accounts for soil buoyancy and a capacity factor that evolves from $N_c = 4.5$ at shallow embedment to $N_c = 10.5$ at an embedment depth when a full-flow mechanism develops. This transitional embedment depth increases with the dimensionless strength, $s_u/\gamma'D$, and consequently the shallow embedment interpretation becomes more important in over consolidated soils.

Also shown on Figure 4 are $s_u$ profiles determined using the classical expression for overconsolidated soil strength (Ladd et al. 1977):

$$s_u = \sigma'_v \left( \frac{s_u}{\sigma'_v}_{\text{nc}} \right) \text{OCR}^\Lambda$$

where $\sigma'_v$ is the current vertical effective stress determined from the profile of effective unit weight, $\gamma'$, with depth established from post-testing sample cores and the (slightly) varying acceleration level with sample depth, and $\Lambda$ is the plastic volumetric strain ratio (Schofield and Wroth 1968). The best
fit between Equation 1 and the experimental $s_u$ profiles on Figure 4 was obtained using $(s_u/\sigma'_{v.local}) = 0.14$ to 0.18 (increasing slightly with centrifuge spinning time) which is typical for centrifuge kaolin samples (Chow et al. 2014; Morton et al. 2014; Hu et al. 2014) and $\Lambda = 0.85$. The latter is at the upper end of the typical range, $\Lambda = 0.7$ to 0.9 (Mitchell and Soga, 2005), and is perhaps reflective of the limited swelling period between scraping the soil surface and the subsequent T-bar test. The theoretical $s_u$ profiles may alternatively be expressed as $s_u = s_{um} + kz$, where the mudline strength intercept, $s_{um} = 0$ to 4 kPa and $k = 0.9$ to 1.2 kPa/m (depending on OCR).

**Interpretation of accelerometer data**

The linear acceleration, $a_{linear}$, due to the motion of the anchor may be deduced from the accelerometer measurements (that also include a component of centripetal acceleration) following the methodology outlined by O’Loughlin et al. (2014b). Figure 5 plots profiles of anchor velocity, $v$, and linear acceleration, $a_{linear}$, with displacement, $z$, where $v$ and $z$ were calculated by numerically integrating $a_{linear}$ (once for velocity and twice for displacement). As recommended by O’Loughlin et al. (2014b), the mudline was established by matching the measured anchor embedment depth and the anchor displacement calculated from the numerical integration. The total anchor displacement was calculated as 416 and 415 mm, which corresponds to free-fall distances of 306 mm (Test 2) and 330 mm (Test 15) compared with the respective preselected drop heights of 308 mm and 332 mm. These small variations were anticipated and are due to extension of the sacrificial release cord as the centrifuge spins up to 100 g. Confidence in the motion data derived from the MEMS accelerometer measurements may also be drawn from the good agreement with the independent velocity measurements obtained from the PERPs.

Figure 5 shows that the theoretical linear acceleration of the anchor (based on the anchor acceleration relative to the sample surface) ignoring friction that develops at the anchor-guide interfaces overestimates the anchor linear acceleration and velocity by up to 35 and 38 % respectively. The actual linear acceleration of the anchor can be estimated by discounting the ‘zero-friction’ linear acceleration, $r\omega^2$, by an amount due to the friction along the guide, $2\mu v$, where $\mu$ is the dynamic
The coefficient of friction (taken as 0.45 for brass/aluminium sliding on steel) and v is the anchor velocity (Chikatamarla et al. 2006).

The resulting acceleration profile obtained using this adjustment is also shown on Figure 5. The agreement with the experimental data is good, with the exception of approximately 40 mm after release where additional resistance (perhaps from the sequential parting of the strands of the release cord, which hindered instantaneous release) causes the linear acceleration derived from the measurements to be lower. Nevertheless, the theoretical linear acceleration satisfactorily quantifies the frictional resistance due to the guide that must be considered over the first anchor length of soil penetration as discussed later in the paper.

**Anchor embedment model**

Data such as those presented on Figure 5 are important for the validation of models for predicting the embedment of dynamically installed anchors. This study considers an anchor embedment model first proposed by O’Loughlin et al. (2004b). Details of this model are provided, before examining its merit by comparing predictions with the centrifuge data considered here.

**Model formulation**

The model considers the forces acting on the anchor in one-dimensional space (depth) and time and equates the net force on the anchor, $F_{net}$, to the product of the anchor mass and acceleration. This is essentially Newton’s second law of motion, which for the anchor problem considered here may be expressed as:

$$F_{net} = (m + m') \frac{d^2z}{dt^2} = W - F_b - R_f(F_{frict} + F_{bear}) - F_d$$

where $m$ is the anchor mass, $m'$ is the added mass of soil that is accelerated with the anchor, $z$ is anchor depth, $t$ is time, $W$ is the anchor dry weight, and the resistance forces acting on the anchor (shown in Figure 6) are: the buoyancy force, $F_b$, equal to the weight of the soil displaced by the anchor, frictional resistance along the soil anchor interface, $F_{frict}$, bearing resistance on the anchor tip and flukes, $F_{bear}$, and drag resistance, $F_d$. The added mass, $m'$ in Equation 2 is an important
consideration for geometries with low aspect ratios (e.g. spheres, Morton et al. 2015) but for long slender bodies such as the anchor considered here, m’ is negligible and can be taken as zero (Beard 1981, Shelton et al. 2011).

As will be described shortly, the $R_f$ term in Equation 2 accounts for the viscous enhancement of strength due to strain rate effects, but does not describe the pressure drag component of drag resistance, which is independent of viscous effects and is linked to the stagnation pressure (Zhu and Randolph 2011; Randolph and White 2012; Blake and O’Loughlin 2015). This drag resistance is well known in fluid mechanics but has also been shown to be an important – and in some instances a dominating – component of resistance for geotechnical problems (Zhu and Randolph 2011; Randolph and White 2012; Sahdi et al. 2014a; Morton et al. 2015). Drag resistance is formulated as:

$$
F_d = \frac{1}{2} C_D \rho_s A_p v^2
$$

where $C_D$ is a drag coefficient, $\rho_s$ is the soil density, $A_p$ is the anchor’s projected area and $v$ is the anchor velocity.

Frictional and bearing resistances can be expressed in the form:

$$
F_{frict} = \alpha s_u A_s; \quad F_{bear} = N_c s_u A_p
$$

where $\alpha$ is an interface friction ratio (of limiting shear stress to undrained shear strength), $N_c$ is the bearing capacity factor for the anchor tip or fluke and $s_u$ is the undrained shear strength averaged over the appropriate contact area – frictional area, $A_s$, or projected area, $A_p$.

As mentioned previously, the high anchor penetration velocity leads to strain rates in the soil that are beyond that associated with undrained behaviour and increase soil strength (Casagrande and Wilson 1951; Graham et al. 1983; Sheahan et al. 1996). As shown by Equation 2, these viscous strain rate effects are accounted for by employing a strain rate function, $R_f$, to scale the undrained shear strength according to the shear strain rate. $R_f$ is generally described using either power or semi-logarithmic functions. The power law is generally preferred for problems involving higher orders of magnitude variation in strain rate (Biscontin and Pestana 2001; Abelev and Valent 2009; O’Loughlin et al. 2013) and may be formulated as:
where \( \dot{\gamma} \) is the strain rate, \( \dot{\gamma}_{\text{ref}} \) is a reference strain rate associated with the measurement of undrained strength and \( \beta \) is a strain rate parameter. Although quantifying the absolute magnitude of strain rate and its variation within the shear zone is non-trivial, it is reasonable to assume that at any given location, the operational strain rate is proportional to \( v/D \), such that:

\[
R_f = \left( \frac{v/D}{(v/d)_{\text{ref}}} \right)^\beta
\]

where \( (v/d)_{\text{ref}} \) in this instance is calculated using the T-bar penetration velocity, \( v \), and diameter, \( d \).

A further aspect of Equation 2 relates to the question of hole closure in the wake of the anchor during dynamic penetration. As discussed by O’Beirne et al. (2015a) and shown by Figure 6, if the hole formed by the passage of the anchor closes immediately after penetration, additional reverse end-bearing resistance should be included at the rear of the anchor fluke and shaft and the soil buoyancy, \( F_b \), should be limited to that calculated using the volume of the anchor (i.e. not considering additional volume from an open shaft hole above; Figure 6, option 2). Conversely, an open hole would require soil buoyancy to be calculated using a volume equal to that of the anchor plus a cylinder with a height equal to the distance from the rear of the anchor to the mudline (assuming the slots formed by the flukes will always close owing to their low thickness, O’Loughlin et al. (2013) and not include additional reverse end bearing resistance at the rear of the anchor (Figure 6, option 1).

As shown by Morton et al. (2014) for pushed-in penetrometers and O’Beirne et al. (2015a) for dynamically penetrating projectiles, the cylindrical hole formed by the passage of a cylindrical or spherical body is expected to close at a transitional depth, \( z_{\text{deep}} \), controlled by the dimensionless strength ratio, \( s_u / \gamma' D \), where \( s_u \) and \( \gamma' \) are the local undrained shear strength and effective unit weight at the rear of the body. This ‘hole-closure depth’, \( z_{\text{deep}}/D \), may be quantified from

\[
\frac{z_{\text{deep}}}{D} = a + \left( b \frac{s_u}{\gamma' D} \right)^c + \frac{d - a}{1 + [(s_u / \gamma' D)/e]^f}
\]

where the fitting constants \( a = 16.3 \), \( b = 0.12 \), \( c = 1.3 \), \( d = 0.52 \), \( e = 4.9 \) and \( f = 1.5 \).
Equation 7 may be used in an embedment prediction by firstly making an assumption regarding hole closure and modelling buoyancy and bearing resistance accordingly. The calculated final embedment depth at the rear of the anchor would then be used to calculate $z_{\text{deep}}/D$ and in turn, to check the validity of the initial assumption, allowing a revised calculation (using the opposing hole closure condition) to be performed if necessary.

**Parameter selection**

The interface friction ratio, $\alpha$, required for calculating the frictional resistance in Equation 4 is identical to that used in the estimation of frictional resistance on driven piles and suction caissons and may be estimated as the inverse of the soil sensitivity. For the kaolin considered in this study, the soil sensitivity is 2.5 as quantified using cyclic T-bar penetrometer tests (e.g. Sahdi et al. 2014b; Colreavy et al. 2016), such that $\alpha = 0.4$.

The capacity factor, $N_c$, used in Equation 4 for the calculation of bearing resistance for the anchor tip and flukes, is similar to that used in the analysis of driven piles and suction installed caissons. American Petroleum Institute guidelines recommend $N_c = 9$ (API, 2002), although for the ellipsoidal tip considered here, static (nominally undrained) penetration tests suggest that $N_c = 12$ is more appropriate (O’Loughlin et al. 2009). The anchor flukes may be considered similar to the skirt tip of a caisson, which is typically modelled using $N_c = 7.5$, analogous to a deeply embedded strip foundation (Skempton 1951).

The anchor drag coefficient, $C_D$, is geometry dependent and may be assessed experimentally (e.g. Freeman and Hollister 1989; Fernandes et al. 2006; Shelton et al. 2011; Hasanloo et al. 2012; Blake and O’Loughlin 2015) or numerically (Raie et al. 2009; Strum et al. 2010; Nazem et al. 2012). Øye (2000) employed computational fluid dynamics to derive $C_D = 0.63$ for the anchor geometry considered here, which compares well with $C_D = 0.67$ as determined from free-fall in water tests using a 1:20 reduced scale model of the same geometry (O’Beirne et al. 2016). The latter value was adopted in this study.

The reference value, $(v/d)_{\text{ref}}$ used in Equation 6 is that associated with the measurement of the undrained shear strength, $s_u$. As presented earlier in the paper, $s_u$ was determined from T-bar
penetrometer tests involving a T-bar diameter, \( d = 5 \text{ mm} \) and a penetration velocity, \( v = 1 \text{ mm/s} \), such that \( (v/d)_{\text{ref}} = 0.2 \text{ s}^{-1} \), which is four orders of magnitude lower than the maximum value associated with anchor penetration, \( v/D = 16.5/0.006 = 2750 \text{ s}^{-1} \).

The strain rate parameter, \( \beta \), is best selected based on strain rate dependency observed in variable rate penetrometer tests, as like the case for a dynamically installed anchor, these tests include compensating effects of strain softening that limit the strength increases associated with increasing strain rates. Therefore, \( \beta \) values should be guided by those measured in variable rate penetrometer tests, which typically give \( \beta = 0.05 \) to 0.09 (Low et al. 2008; Chung et al. 2006), and are approximately equivalent to a 12 to 21\% increase in strength per decade increase in strain rate. A mid-range \( \beta = 0.07 \) was initially assumed for the embedment prediction model.

These model parameters are listed in Table 2.

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**Application to centrifuge test data**

As outlined earlier in the paper, continuous motion data captured during dynamic embedment permit for a rigorous assessment of anchor embedment models in accurately describing the processes associated with dynamic penetration. In the following sections, experimental profiles of acceleration and velocity with penetration depth are compared with corresponding theoretical profiles.

Equation 2 is appropriate for field cases, but needs to be extended slightly in modelling centrifuge tests to account for the friction between the anchor and the guide, as this represents an additional component of resistance over the first anchor length of penetration (less if the anchor guide is offset from the mudline, as is the case for the centrifuge tests considered here). This was accounted for in the model predictions by including an extra resistance due to the guide friction, equal to \( 2\mu \omega \nu m \) (as discussed earlier in the paper) when the anchor was fully within the guide, but diminishing in a linear fashion as the length of anchor in the guide reduced to zero.

The model was initially applied to Test 2 using the initial parameter set listed in Table 2. A comparison of the measured and predicted velocity and acceleration profiles is provided on Figure 7, where it is evident that the embedment model over-predicts the final embedment depth, \( z_{\text{e,tip}} \), by
approximately 20%. Over-prediction of the embedment depth suggests that the resistance forces acting on the anchor during soil penetration are underestimated. O’Loughlin et al. (2009) showed that the parameter set in Table 2 is appropriate for modelling the penetration resistance of dynamically anchors under nominally undrained conditions (i.e. excluding the parameters $C_D$ and $\beta$ that account for drag and strain rate effects). As such, the under-prediction of the total penetration resistance – evident from Figure 7 – is considered to be due to higher strain rate effects than allowed for using Equation 6 and the parameters listed in Table 2.

**Back calculated strain rate factor**

As demonstrated in O’Loughlin et al. (2016), measurement of the anchor acceleration allows the strain rate factor, $R_f$, to be quantified by rearranging Equation 2 to give:

$$R_f = \frac{W - m\frac{d^2z}{dt^2} - F_b - F_d}{F_{frict} + F_{bear}}$$

where $d^2z/dt^2$ now represents the linear acceleration derived from the MEMS accelerometer measurements, and $W$, $F_b$, $F_d$, $F_{frict}$ and $F_{bear}$ are known or formulated quantities using the parameters outlined previously (and listed in Table 2). The experimental $R_f$ determined using Equation 8 are shown against velocity on Figure 8 for Tests 2, 7 and 15 (refer to Table 1), reflecting the range of strength profiles in Figure 4. Also shown on Figure 8 is $R_f$ formulated using the power law (Equation 6) using $\beta = 0.07$ (i.e. as listed in Table 2 and used for the prediction on Figure 7), $\beta = 0.13$ and a varying $\beta$, discussed further below.

The three tests considered on Figure 8 demonstrate a consistent variation in the strain rate factor $R_f$ with anchor penetration velocity, $v$, with $R_f \sim 4$ at the highest anchor velocity, $v \sim 17$ m/s, where the experimental data appear noisier due to wider variations in the measured acceleration at impact with the mudline and during shallow penetration. $R_f \sim 4$, means that the mobilised geotechnical resistance is 4 times higher than would be mobilised under nominal undrained conditions, demonstrating the importance of strain rate effects in dynamic penetration problems. However, as discussed in the
following, strain rate effects are more pronounced in these centrifuge tests than in equivalent field conditions.

Also shown on Figure 8 is the theoretical variation in $R_f$ using Equation 8, with variations in the magnitude of $\beta$. O’Loughlin et al. (2013) and Blake and O’Loughlin (2015) suggest that for dynamic penetration problems, $\beta$ should be similar to that determined from variable rate penetrometer testing, provided that the maximum strain rate in the anchor tests is no more than three orders of magnitude higher than the strain rate associated with the reference soil strength. This is because over several orders of magnitude increase in strain rate, the mobilised shear strength increases more rapidly than can be fitted using a power law (Biscontin and Pestana 2001; Peuchen and Mayne 2007; Lunne and Andersen 2007; Jeong et al. 2009). For these centrifuge tests, where the velocity is unscaled (relative to field conditions), whilst the diameter is scaled by a factor of 100, the maximum (proxy) strain rate is $v/D = 16.5/0.006 = 2750$ s$^{-1}$, which is four orders of magnitude higher than $(v/d)_{ref} = 0.2$ s$^{-1}$. Consequently, and as shown by Figure 7, using $\beta = 0.07$ with Equation 6 significantly underestimates $R_f$. A higher operational $\beta = 0.13$ provides a better overall match with the experimentally derived $R_f$, consistent with back analysed $\beta$ values from dynamically installed anchor centrifuge tests reported by O’Loughlin et al. (2013). However, the predictions under-predict strain rate dependency as anchor velocity increases beyond $v \sim 12$ m/s and over-predict strain rate dependency as anchor velocity decreases beneath $v \sim 8$ m/s.

A more reasonable approach would be to allow $\beta$ to retain a lower constant value (within $\beta = 0.06$ to 0.08) over the first two orders of magnitude increase in $v/D$ above the reference $(v/d)_{ref}$, but to then gradually increase with increasing $v/D$. This may be obtained by formulating $\beta$ according to:

$$\beta = \beta_{min} + \frac{(\beta_{max} - \beta_{min})}{1 + \frac{(v/D)_{50}}{v/D}},$$

where $\beta_{min}$ is a lower bound on $\beta$ (in the range 0.06 to 0.08), $\beta_{max}$ is a limiting strain rate parameter and $(v/D)_{50}$ represents the value of $v/D$ at which $\beta$ is the average of $\beta_{min}$ and $\beta_{max}$. The resulting formulated $R_f$ described using Equations 8 and 9 with $\beta_{min} = 0.07$, $\beta_{max} = 0.17$ and $(v/D)_{50} = 1000$ is seen on Figure 8 to describe the variation in strain rate dependence adequately, despite slight over predictions at very low velocities, $v < 1$ m/s.
Model predictions

In this section theoretical profiles of acceleration and velocity with penetration depth are compared with corresponding experimental measurements for four tests from the dataset considered here, representing the range of soil strength profiles on Figure 4. Modifications were made for each prediction ‘scenario’ to investigate the influence of the resistance components on Figure 6. The prediction cases and model parameters are listed in Table 3 and investigate scenarios related to: (i) selection of strain rate parameter, (ii) hole closure in the wake of the anchor, (iii) inclusion of drag resistance and (iv) basis for modelling frictional resistance.

The results of these predictions are compared with the corresponding experimental data for the four considered tests in Figures 9, 10, 11 and 13 for scenarios (i), (ii), (iii) and (iv) respectively and are discussed in the following sections.

Selection of strain rate parameter

The 20% over-prediction of the final anchor embedment depth on Figure 7, and back-analysis of $R_f$ on Figure 8, suggests that when the strain rate variations exceed three orders of magnitude, soil strength increases much more rapidly than can be described using the power strain rate law, allowing for a typical ~15% increase in soil strength per decade increase in strain rate ($\beta = 0.07$ corresponds to a ~15% increase in strength per decade increase in strain rate). The average operable soil strength estimated using $\beta = 0.13$ is closer to a 30% increase per log cycle of strain rate, and indeed Figure 9 shows that allowing for a higher $\beta (= 0.13$; prediction A) leads to final embedments that are within ±1% of the measurements. However, the match between the measured and predicted acceleration profiles improves using $\beta$ according to Equation 9 (prediction B), particularly at deeper embedments where $v/D$ is within three orders of magnitude of $(v/d)_{ref}$ and Equation 9 returns $\beta$ to close to $\beta_{\text{min}} = 0.07$. For field cases the maximum strain rate is expected to be of the order of 20 to 30 s$^{-1}$, which is only two orders of magnitude higher than the strain rate associated with in-situ measurement of soil strength using a cone, T-bar or ball penetrometer ($v/d = 0.25$ to 0.6 s$^{-1}$ for typical penetrometer diameters and a penetration velocity = 20 mm/s). Using Equation 9 in this case would maintain $\beta = \beta_{\text{min}}$ such that Equation 9 is unlikely to be required for field conditions. Caution should be exercised if
a soil strength measured in laboratory element tests at much lower strain rates (typically 1%/hr) is adopted as the reference soil strength in the power law, as this would result in strain rate variations that span seven orders of magnitude. In such cases the intact laboratory strength should be adjusted to reflect the strain softening associated with anchor installation and the strain rate dependency needs to be more moderate over the first few decades increase in strain rate.

**Hole closure**

Figure 10 compares measured acceleration and velocity profiles with prediction C, which assumes an open hole in the wake of the anchor, and prediction D, which assumes a closed hole. For these tests the maximum anchor embedment depth was 1.47 anchor lengths, such that the difference between modelling an open or closed hole only arises close to the final embedment depth. However, the measured and predicted profiles for $z > L$ are in better agreement when the hole formed by the advancing anchor is modelled according to Equation 7, which predicts a closed hole for Tests 2, 4 and 7 and an open hole for Test 15. Although the maximum effect on the final predicted embedment depth is less than ± 2% for the anchor geometry considered here, more pronounced effects would be expected on objects with lower aspect ratios, such as some designs of free-falling penetrometers for rapid measurement of seabed soil strength (e.g. Stark et al. 2009; Morton et al. 2015).

**Drag resistance**

Figure 11 shows that the inclusion of drag resistance during soil penetration is appropriate (prediction B), as the agreement between the measured and theoretical profiles is much poorer when it is excluded (prediction E). Attempting to compensate for the lower resistance due to exclusion of drag resistance required an interface friction ratio, $\alpha = 0.51$ to 0.57 (prediction F), which is incompatible with the remoulded soil strength and still gave a much poorer match to the measurements than was achieved when drag was included and the interface friction ratio was taken as the inverse of the soil sensitivity ($\alpha = 0.4$).
Frictional resistance

Some studies on the dynamic penetration of cylindrical penetrometers have concluded that strain rates are higher for frictional resistance than for bearing resistance (e.g. Dayal et al. 1975; Steiner et al. 2014; Chow et al. 2014; O’Loughlin et al. 2016). This can be accommodated in a power function by including a coefficient, n (Einav and Randolph 2006), such that Equation 6 becomes:

\[ R_f = \left( n \frac{v/D}{(v/d)_{ref}} \right)^\beta \]  

10

Bearing resistance is modelled using n = 1 (Zhu and Randolph, 2011), whereas frictional resistance may be modelled either with similar strain rate dependency by also adopting n = 1, or by considering n as a function of \( \beta \) (adapted from Einav and Randolph 2006):

\[ n = 2 \left( \frac{n_i}{\beta} + n_1 - 2 \right) \]  

11

where \( n_i = 1 \) for axial loading, i.e. relevant to the ‘axial penetration’ of a dynamically installed anchor. Hence for \( \beta = 0.07 \) (typical for field conditions) strain rate enhancement of frictional resistance would be approximately 27 times higher than for bearing resistance.

Similar to the methodology adopted for Figure 8, strain rate factors for bearing and frictional resistance – \( R_{f,\text{bear}} \) and \( R_{f,\text{frict}} \) respectively – were determined according to:

\[ R_{f,\text{bear}} = \frac{W - m \frac{d^2 z}{dt^2} - F_b - F_d}{n^\beta F_{f,\text{frict}} + F_{\text{bear}}} \]  

12

\[ R_{f,\text{frict}} = n^\beta R_{f,\text{bear}} \]  

13

where \( \beta \) was allowed to vary with v/D according to Equation 9. \( R_{f,\text{bear}} \) and \( R_{f,\text{frict}} \) quantified using Equations 12 and 13 are shown against velocity on Figure 12, together with the theoretical variation in \( R_f \) according to Equation 11. The agreement between the experimental data and Equation 10 shown on Figure 12a is clearly quite poor, suggesting that higher strain rate dependency on friction resistance is not warranted. However, a second set of comparisons shown on Figure 12b, which were obtained by lowering the interface friction ratio from \( \alpha = 0.4 \) (Figure 12a) to \( \alpha = 0.29 \), produces agreement that
is equivalent to that obtained using $\alpha = 0.4$, but assuming the same strain rate dependency for friction and bearing (see Figure 9). This is made clearer by the predicted and experimental anchor motion profiles on Figure 13. Figure 13 compares predictions obtained assuming higher strain rate dependency for frictional resistance, with $\alpha = 0.4$ (prediction G) and with $\alpha = 0.29$ (prediction H), with the reference predictions that were obtained assuming the same strain rate dependency for friction and bearing and $\alpha = 0.4$ (prediction B). As expected from Figure 12a, prediction G is in poor agreement with the measurements. Predictions B and H are in very good agreement with the measurements and are essentially coincident.

To explore this further, the embedment model using prediction scenarios B and H was applied to a large database formed from the tests considered here (and listed in Table 1) and tests reported by O’Loughlin et al. (2013). The O’Loughlin et al. (2013) data relate to centrifuge tests in normally consolidated clay at 200 g using an anchor with the same geometry as that considered here, but with varying mass and with and without flukes. The comparisons are made on Figure 14 in terms of the ratio of predicted to measured final anchor embedment as measurements in the O’Loughlin et al. (2013) tests were limited to the impact velocity and final embedment depth. The initial observation from Figure 14 is that the embedment model is capable of predicting the entire database (114 anchor installations) to within ±15% of the measurements, albeit that $(v/D)_{50} = 5000$ was required for the O’Loughlin et al. (2013) tests, consistent with the slightly lower (on average) reported $\beta$ values reported in that study. The second observation is that consistent with Figure 13, essentially identical predictions are obtained using prediction B (same strain rate dependency for friction and bearing and $\alpha = 0.4$) and prediction H (higher strain rate dependency for frictional resistance (Equations 10 and 11) and $\alpha = 0.29$). Finally, Figure 14 shows that the prediction accuracy does not exhibit any bias with average strain rate, as quantified by the horizontal axis of Figure 14, $(v/D)_{av}$.

A reduced $\alpha$ is quite reasonable if water becomes entrained at the anchor-soil interface, as is quite possible for dynamic penetration problems. This has also been observed in high-rate ring-shear tests (Tika and Hutchinson 1999) where water was permitted to penetrate the shear zone, in suction caisson installations where water became trapped between internal stiffeners (Gaudin et al. 2014) and during repeated cyclic penetration of risers from above the mudline to within the seabed (Yuan et al. 2016).
Although the data considered here are inconclusive as to the question as to whether strain rate effects are higher for frictional resistance, experimental data on free-fall cone penetrometers instrumented with both tip and shaft load cells are persuasive in this regard (Steiner et al. 2014). This suggests that an allowance for higher strain rate dependence on frictional resistance may be justified, albeit that in some instances the increased frictional resistance may be compensated by a reduced mobilised remoulded soil strength associated with water entrainment.

**Concluding remarks**

Dynamically installed anchors may be a cost effective technology for mooring floating facilities, particularly mobile operations, but also risers and pipelines. Predicting their capacity is relatively straightforward (e.g. O’Loughlin et al. 2004a; O’Beirne et al. 2015b) provided their embedment depth can be predicted reliably. This is challenging as it requires an assessment of soil shearing at extremely high strain rates, consideration of drag resistance and other potential hydrodynamic effects such as entrainment of water along a boundary layer at the anchor wall. This paper considers the installation problem by making comparisons of anchor motion data obtained from centrifuge tests in normally and over consolidated clay with predicted responses, as obtained using an analytical embedment model cast in terms of strain rate enhanced shear resistance and drag resistance.

Comparisons of the measured and predicted anchor motion response reveal that the embedment model is capable of describing the anchor motion accurately in both normally and over consolidated clay. Scaling the undrained soil strength to account for the very high strain rates and inclusion of drag resistance were seen to be important considerations. The scaling in the centrifuge tests resulted in strain rates that were about four orders of magnitude higher than the strain rate associated with the measurement of the reference undrained shear strength, and about two orders of magnitude higher than equivalent strain rates in the field. The wider range of strain rates in the centrifuge tests revealed a limitation in the power strain rate law that was adopted here for scaling the soil strength according to the strain rate (although an equivalent limitation would also apply to the semi-logarithmic law). This required the strain rate parameter in the power law to evolve with the mobilised strain rate and a simple backbone curve was proposed in the paper to cater for this.
Predictions of a database of over 100 anchor installations – comprising the centrifuge tests reported in this paper and also centrifuge tests reported by O’Loughlin et al. (2013) – resulted in an accuracy in the final anchor embedment depth that was within ±15% of the measurements. These predictions also revealed that there may be a higher strain rate dependency on frictional resistance than on bearing resistance, although this required the adoption of a reference soil strength that was lower than the fully remoulded strength, suggesting the entrainment of water at the anchor-soil interface.

Acknowledgements
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References


**Figure Captions**

Figure 1. Dynamically installed anchors: (a) Deep Penetrating Anchor (Deep Sea Anchors 2009), (b) installation procedure (after Lieng et al. 2000).

Figure 2. Dynamically installed model anchor featuring an elliptical tip and four large flukes (modelled on an equivalent prototype anchor; Lieng et al. 1999).

Figure 3. Centrifuge testing arrangement.

Figure 4. Undrained shear strength and overconsolidation ratio profiles for scrape amounts ($\Delta z_{\text{scrape}}$) of: (a) 0 mm, (b) 15 mm, (c) 25 mm, (d) 35 mm, (e) 45 mm, (f) 55 mm.

Figure 5. Interpretation of the MEMS accelerometer data: velocity profile during descent through guide and embedment in soil for: (a) Test 2, $s_{um} = 0$ kPa (b) Test 15, $s_{um} = 4.1$ kPa.

Figure 6. Resistance forces acting on a dynamically installed anchor during installation: option (i) for an open hole in the wake of the anchor and option (ii) for a closed hole in the wake of the anchor.

Figure 7. Comparison of experimental and predicted velocity and linear acceleration profiles for Test 2 using the model parameters given in Table 2.

Figure 8. Assessing the performance of the power law in describing the experimentally quantified strain rate dependence.

Figure 9. Evaluating the merit of allowing the strain rate parameter, $\beta$, to evolve with mobilised strain rate relative to an average ‘operable’ $\beta$: (a) Test 2, (b) Test 4, (c) Test 7 and (d) Test 15.

Figure 10. Examining the influence of modelling the hole formed in the wake of the anchor as fully closed or fully open: (a) Test 2, (b) Test 4, (c) Test 7 and (d) Test 15.

Figure 11. Assessing the influence of drag resistance on the anchor motion response: (a) Test 2, (b) Test 4, (c) Test 7 and (d) Test 15.

Figure 12. Evaluation of the power law with allowance for higher strain rate effects on frictional resistance (Equations 10 and 11) assuming: (a) $\alpha = 0.4$ (b) $\alpha = 0.29$.

Figure 13. Examining the influence of higher strain rate dependency for frictional resistance: (a) Test 2, (b) Test 4, (c) Test 7 and (d) Test 15.

Figure 14. Predicted against measured final anchor embedment depths assuming the same strain rate dependency for frictional and bearing resistance with $\alpha = 0.4$ (Prediction B) and higher strain rate dependency for frictional resistance with $\alpha = 0.29$. 
Table 1 Summary of test results

<table>
<thead>
<tr>
<th>Test</th>
<th>Mudline, $s_{um}$ (kPa)</th>
<th>Gradient with depth, $k$ (kPa/m)</th>
<th>Impact velocity, $v_i$ (m/s)</th>
<th>Tip embedment depth, $z_{tip}$ (mm)</th>
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$^1$ Estimated value from PERPs data – MEMS accelerometer data not captured
<table>
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<td>Strain rate parameter, $\beta$</td>
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<td>Anchor drag coefficient, $C_D$</td>
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<td>Bearing capacity factor, $N_c$</td>
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<td>Reference strain rate, $(v/d)_\text{ref}$ (s$^{-1}$)</td>
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<td>Hole closure</td>
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Table 3 Model parameters adopted in the various prediction scenarios (modifications to the values/approach given by Table 2)

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<th>Prediction scenario</th>
<th>Prediction</th>
<th>( \beta )</th>
<th>( \beta_{\text{min}} )</th>
<th>( \beta_{\text{max}} )</th>
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<th>( n_l )</th>
<th>Hole closure</th>
<th>( C_d )</th>
<th>( \alpha )</th>
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<td>(i) Selection of strain rate parameter (Figure 9)</td>
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<td>Equation 9</td>
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<td>Equation 11</td>
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Strain rate factor, $R_f$

Velocity, $v \text{ (m/s)}$

- Equation 10
- Test 2
- Test 7
- Test 15

(a)
Figure 12 Evaluation of the power law with allowance for higher strain rate effects on frictional resistance (Equations 10 and 11) assuming: (a) $\alpha = 0.4$ (b) $\alpha = 0.29$
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