Installation and Monotonic Pullout of a Suction Caisson Anchor in Calcareous Silt

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ABSTRACT

This paper reports results from a series of model tests undertaken to provide insight into the behaviour of a stiffened caisson anchor during installation and monotonic pullout in lightly overconsolidated calcareous silt. The tests were carried out in a beam centrifuge, varying the installation process (jacked in or jacked in followed by suction assisted installation), and the consolidation period prior to anchor pullout. The mudline load inclination was also varied to encompass various mooring configurations.

The centrifuge data were used to calibrate analytical installation and vertical pullout capacity models, based on conventional bearing and frictional resistance factors but with strain rate dependent and strain softened undrained shear strength for the soil. Piezocone based direct design approaches were also proposed, deriving end bearing and frictional resistance from cone tip resistance and sleeve friction, respectively. Holding capacity of suction caisson under inclined loading was presented as failure envelopes expressed in terms of dimensionless vertical and horizontal components of caisson net resistance, which agreed well with a plastic limit analysis based envelope developed for suction caissons in clay. The regain of anchor capacity was found to be in good agreement with predictions based on the cavity expansion framework.

KEYWORDS: anchors; calcareous soils; clays; centrifuge modelling; failure; offshore engineering
Suction caissons were first used some 30 years ago and nowadays are the most widely used anchor for floating facilities in deep water. They have been proven as a cost-effective alternative to more traditional anchoring solutions such as piles and drag anchors. In addition to use as anchors for floating facilities, e.g. floating production systems, tension leg and SPAR platforms, they are also used extensively for pipeline manifolds and end terminals, and as the foundation for riser towers (e.g. Sparrevik, 2002).

Suction caissons are large diameter steel cylinders, open ended at the bottom and closed at the top, typically 3~30 m long and with an aspect ratio (length to diameter, L/D) in the range 2~7 (Andersen et al., 2005). These are thin-walled (thickness, $t \leq 50$ mm), with D/t exceeding 150, and thus prone to buckling during installation or distortion under the anchoring load. As such, horizontal ring stiffeners at intervals along the inner wall of the thin skirt are employed with local thickening of the wall in the vicinity of the padeye, with or without transverse struts.

The caisson is installed by pumping water from inside the caisson after it is allowed to penetrate under its self-weight. The difference between the hydrostatic water pressure outside the cylinder and the reduced water pressure inside provides a differential pressure, or suction, that acts as a penetration force (Andersen et al., 2005). The padeye of the installed caisson is linked to the floating facility by a mooring line (chain). Operational loadings are commenced after a set-up period, during which the soil regains strength. Operational loading angle (and padeye position) is critical and varies with mooring type. For catenary and taut leg mooring the padeye remains below the seabed and the mudline load inclinations ($\theta$) are respectively $10^\circ$~$25^\circ$ and $30^\circ$~$60^\circ$ to the horizontal (Figure 1a). For tension leg platforms the padeye is attached at the top of a caisson and the loading is quasi-vertical.
The majority of deep water developments, mooring floating facilities by caisson anchors, have occurred in clayey seabeds. As such, suction caissons are now a mature product in clay, with well developed analysis procedures covering both installation and loading (Andersen et al., 2005; Randolph et al., 2011). Historically, examples of notable contributors to this area are Andersen, Clukey, Colliat, Dendani, and Jostad who reported field data and shared their experiences and thoughts (e.g. Dendani & Colliat, 2002; Dendani, 2003; Andersen et al., 2005). For installation, recent advances have been resulted from FE analysis, model testing and field trials (e.g. Andersen & Jostad, 2002; Randolph & House, 2002; Andersen & Jostad, 2004; Jeanjean et al., 2006; Zhou & Randolph, 2006; Chen & Randolph, 2007; Vásquez et al., 2010; Gaudin et al., 2014; Zhou et al., 2016). Design methods for assessing caisson penetration resistance or underpressure required for caisson installation were given by Andersen & Jostad (1999); Houlsby & Byrne (2005). Capacity of suction caissons under monotonic vertical and inclined loadings has been addressed through centrifuge model tests, FE analyses and upper bound solutions (e.g. Andersen & Jostad, 1999; Deng & Carter, 2002; Randolph & House, 2002; Aubeny et al., 2003a, 2003b; Supachawarote et al., 2004; Vásquez et al., 2010). Recommendations for installation, operation and removal in clay are provided by API RP 2SK (2005) and DNV (2005).

However, little data exist for caisson in calcareous soils, which are prevalent in many seabed deposits, particularly in the oil and gas producing offshore regions of Australia. It is identified that only a few suction caisson anchors installed in calcareous seabed, compared to more than 500 caissons (as of 2005) installed around the world mostly in clayey seabeds in water depths to nearly 2000 m (e.g. the Gulf of Mexico, North Sea, offshore West Africa and Brazil; Andersen et al., 2005). For instance, 9 suction caissons were installed in Australia’s calcareous seabed around 17 years ago for mooring the Laminaria FPSO in 340 m water depth, with the installation data reported by Erbrich & Hefer (2002). Data from centrifuge
model tests include bucket foundations (L/D = 0.4~0.5) on calcareous silt (Watson et al., 2000), and a suction caisson (L/D = 2.35) on silty clay (Randolph et al., 1998). Estimation of caisson behaviour in calcareous deposits, both during installation and operation, is identified as problematic. This is due to the characteristics of the calcareous sediments such as high carbonate content, high in-situ void ratio, and high sensitivity. These factors may potentially affect all the resistance components including mobilised end bearing, operational shear strength, friction factor, and caisson-soil interface behaviour. As such, research outcomes established for clay may not be applicable for calcareous soils, and indeed Erbrich & Hefer (2002) identified significant over estimation of underpressure (or penetration resistance) comparing lower bound estimations using the design methods established for clay and recorded data from caisson installation at the Laminaria field.

This paper addresses this gap by reporting centrifuge data on a stiffened caisson in lightly overconsolidated calcareous silt, addressing the effects of installation method, reconsolidation time and mudline loading angle on caisson performance. The data are used to examine the merit of design tools for predicting underpressure required for caisson installation and capacity under monotonic loading.

2 CENTRIFUGE MODELLING

2.1 Experimental Program

The experimental program comprised centrifuge modelling of installation and monotonic pullout of a stiffened caisson anchor in calcareous silt. The work was carried out at 200 g in the beam centrifuge at the University of Western Australia (Randolph et al., 1991). It has a swinging platform radius of 1.8 m with a nominal working radius of 1.55 m. The platform supports a standard rectangular ‘strongbox’, which has internal dimensions of 650 (length) × 390 (width) × 325 (depth) mm, representing a prototype test bed of up to 130 m long by 78 m
wide by 65 m deep at 200 g. The various scaling relationships for modelling at enhanced accelerations were reported by Schofield (1980) and Garnier et al. (2007).

In a centrifuge the acceleration level increases with radius. The acceleration level of 200 g for testing was set at one-third of the sample height, measured from the sample surface, or half of the caisson installation depth. The influence of the non-uniform acceleration field was accounted for in the interpretation of the caisson tests.

2.2 Model Caisson

Tests were performed using a model stiffened, cylindrical caisson of 40 mm in diameter, 120 mm skirt length, 0.4 mm wall thickness ($D = 8$ m, $L = 24$ m, $t = 0.08$ m in prototype scale at 200 g), as shown in Figure 1b. The model consisted of five $2.5 \times 1$ mm inner ring stiffeners spacing equally at 20 mm centre to centre, with the bottom stiffener centre at a distance of 20 mm from the skirt tip. The model was machined from a full block of duraluminium. Table 1 summarises the anchor geometry in model and prototype scale. The overall geometry and dimensions in prototype scale are similar to the suction caissons used to anchor riser towers at Site C of Block B17, offshore Angola (Colliat et al., 2007), although the skirt was slightly thicker, 0.08 m compared to 0.05 m, due to machining constraints.

A pore pressure transducer (PPT) was set at the caisson lid invert to measure the differential pressure across the lid. The lid of the model was also equipped with a one-way valve (allowed the inside water to expel during caisson installation and for holding suction during pullout) and an opening for the syringe pump to apply suction. The padeye was located at about 0.7L below the caisson lid invert, as was suggested by Randolph & House (2002) and others to achieve the optimum capacity under nearly zero mudline load inclination. These are labelled in Figure 1.
2.3 Sample Preparation

The anchor tests were performed in samples of calcareous silt (LL = 72%; PL = 38%; $G_s = 2.67$). The material was sieved through a 2 mm sieve to remove any shell fragments. A slurry was reconstituted by mixing the sieved material with fresh water at a moisture content of about 130% (about 1.5 times the liquid limit) and subsequently de-airing it under a vacuum.

The slurry was then poured into the beam strongbox lined with a thin drainage sand layer. The strongbox was transferred to the beam centrifuge and the sample was consolidated in-flight at 200 g for five days. T-bar penetrometer tests were then conducted to ensure that the strength profile increased linearly with depth, indicating that consolidation was complete. The top 2 mm was then manually trimmed off at 1 g using a custom designed scraper, taking care to minimise disturbance to the remaining soil. This resulted in a perfectly flat sample surface and a mudline strength greater than 0 kPa. The sample was then respun to 200 g for a period of about 24 hours before commencing further strength characterisation tests as described below. The final thickness of each sample prior to testing was about 200 mm (40 m in prototype scale). At all stages during the tests (consolidation, sample trimming and testing), a layer of water was maintained at the sample surface to ensure saturation. Two box samples were prepared, with the tests on the first box focused mainly on vertical penetration-vertical extraction and those on the other one on vertical penetration-lateral extraction.

2.4 Soil Characterisation

Characterisation tests (as illustrated in Figure 2a) were carried out in-flight using a model T-bar penetrometer, with model dimensions 5 mm in diameter and 20 mm in length (see insert of Figure 2b), and a model piezocone penetrometer, with a diameter of 10 mm in model scale (see insert of Figure 2c). The T-bar and piezocone were penetrated and extracted at $v = 1$ mm/s and 1.15 mm/s respectively, giving a dimensionless velocity $V = vD_{ep}/c_v \sim 300 > 30$. 


(where the average coefficient of consolidation over the penetration distance \(c_v = 1.2 \text{ m}^2/\text{year}\) obtained from separate consolidation tests), ensuring undrained conditions (Finnie & Randolph, 1994). Typical shear strength profiles are plotted in Figure 2a. In addition to accounting for the non-uniform acceleration field with sample depth, for deducing shear strength, White et al.’s (2010) method with a deep bearing factor of \(N_{T-bar} = 10.5\) was used for the measured T-bar resistance and a constant bearing factor of \(N_{kt} = 13.56\) (Low et al., 2010) was used for the piezocone resistance data. The undrained shear strength profiles deduced from the T-bar and piezocone penetration test data are in good agreement, and can be idealised averaging the profiles as \(s_u = s_{u,p} = 1.5 + 1.7z \text{ kPa}\). The profiles were consistent across the two samples and hence are not noted separately. Furthermore, T-bar tests were conducted before and after the anchor tests in each sample, although there were no noticeable differences in the strength profiles over the course of the testing. Effective unit weight profiles derived from the water content measurements increased linearly with sample depth according to \(\gamma' = 5.9 + 0.03z \text{ kN/m}^3\).

The T-bar was extracted after a reconsolidation period of \(t_c = 0\) (immediately after the completion of penetration) and 12 months (in prototype scale), and the piezocone was extracted after \(t_c = 12\) months. The corresponding deduced shear strength profiles can be averaged and idealised as \(s_{ue} = 0.5 + 1.1z \text{ kPa}\) for \(t_c = 0\) month and \(s_{ue} = 0.5 + 1.5z \text{ kPa}\) for \(t_c = 12\) months (see Figure 2a, in which ‘-ve’ indicates extraction).

In addition to the standard T-bar tests, cyclic remoulding T-bar tests were conducted at a sample depth of either 15.7 m or 17.7 m and involved cycling the T-bar by \(\pm 2\) T-bar diameters over around 15 cycles. The results from the cyclic T-bar tests provided insight into the degradation of the soil (with degradation factor signifying the ratio of undrained shear strength or T-bar penetration resistance after each cycle to that at identical depth during first penetration), as plotted against number of cycles in Figure 2b together with the theoretical
degradation response calculated using the method suggested by Einav & Randolph (2005), which indicates that the sensitivity of the soil is $S_t = 4.5 \sim 5.0$.

A piezocone penetration test with pore pressure dissipation (measured at the $u_2$ position i.e. at the piezocone shoulder) was carried out at a sample depth of 15.5 and 22.5 m. The pore pressure dissipation data (see Figure 2d) was used to determine the coefficient of (as appropriate) horizontal consolidation ($c_h$) by matching the test data with the Teh & Houlsby (1991) theoretical solution

$$U = \frac{\Delta u}{\Delta u_{\text{max}}} = (0.85 + 10T^*)^{0.45} - 0.08 \quad \text{where} \quad T^* = \frac{c_h t_d}{\sqrt{I_r R_e^2}}$$

where $U$ is the normalised excess pore pressure and the rigidity index $I_r$ of $\sim 150$ corresponding to the average shear modulus ($G_{50}$) was derived from simple shear test data.

The initial rise in pore pressure in Figure 2d reflects the fact of local equalisation of the high pore pressure gradient around the piezocone tip prior to dissipating (Campanella et al., 1986). However, the pore pressure response agrees well with the theoretical solution for $T^* \geq 0.03$, with the best agreement obtained using $c_h = 7 \text{ m}^2/\text{year}$ (at 15.5 m) and $14.5 \text{ m}^2/\text{year}$ (at 22.5 m).

### 2.5 Caisson Testing Procedure

A total of thirteen caisson installation and pullout tests were performed (see Table 2). The caisson was connected to an actuator that allowed vertical movement under either displacement or load control. Installation was conducted by jacking the caisson (with the lid vented) into the soil at a constant displacement rate of 0.4 mm/s using the displacement-controlled system until the measured penetration resistance was close to the prototype submerged self-weight of the caisson, resulting in a penetration of about 60 mm. Once this load was reached, the actuator was programmed to swap to the load-controlled system and
maintain the load constant. Further installation for Tests T1~T4 (Table 2) was achieved by
extracting the water from inside of the caisson using a syringe pump (see Figure 3), with the
set flow rate equalised suction assisted penetration rate and jacking penetration rate.
Installation by jacking (using the displacement-controlled system) up to the full penetration
depth was carried out for Tests T5~T13 (Table 2).
After caisson installation, the system was kept as load-controlled (for suction installation) or
swapped from displacement-controlled to load-controlled (for jacked installation) to hold the
load for a reconsolidation period. For vertical pullout ($\theta_0 = 90^\circ$, Tests T1~T9; Table 2), a
varying reconsolidation period (0, 2, 3, 6, 12 months; Table 2) was permitted before
extracting the anchor at a rate of $v = 0.4 \text{ mm/s}$ (using the displacement-controlled system) to
measure the monotonic anchor capacity. The same system of installation was used with no
mooring line attached. The valve at the caisson lid was shut, allowing fully sealed extraction.
Except, allegedly carried out vented Tests T4 and T9 to examine the corresponding loss of
capacity. Most of the tests were aborted after achieving the peak capacity to limit the
disturbed zone, while some tests (Tests T5, T6 and T9; Table 2) were continued up to full
extraction.
For inclined loading ($\theta_0 < 90^\circ$, Tests T10~T13; Table 2), the centrifuge was ramped down to
connect the model mooring line to the actuator in preparation for the caisson pullout. The
mooring line connected the padeye to the actuator via a pulley, and the mudline load
inclination was adjusted (see Figure 3). Anchor capacity was measured by a 4 kN load cell
mounted in-line with the mooring line close to the actuator. The centrifuge was then respun to
200 g and a reconsolidation period of 12 months was permitted before extracting the anchor at
a rate of $v = 0.4 \text{ mm/s}$ with the valve at the caisson lid shut.
The penetration rate of 0.4 mm/s, with $V = v(t + b)/c_v \sim 30.5 > 30$, resulting in largely
undrained conditions (Finnie & Randolph, 1994). For sealed extraction (Tests T1~T3, T5~T8,
T10~T13; Table 2), the rate of 0.4 mm/s allowed for achieving the dimensionless velocity, $V = vD/cv \sim 420 > 30$, which was sufficient to ensure undrained conditions. For vented extraction (Tests T4 and T9; Table 2), $V = v(t + b)/cv \sim 30.5 > 30$ ensured largely undrained conditions.

3 RESULTS AND DISCUSSION: INSTALLATION

3.1 Caisson Installation

Installation resistance profiles are summarised in Figure 4, reported as the applied load (for jacking) or underpressure multiplied by the inside cross section area beneath the caisson lid. The profiles are somewhat consistent with little jumps at a penetration depth of ~4 m corresponding to the touchdown of the bottom stiffener base with the mudline and at a depth where the caisson lid touched the surface of the free water layer. The results lead to the following comments.

a) The effect of installation method on the penetration resistance profile including the maximum resistance at the final penetration depth (i.e. the underpressure required for installation) is trivial. This is consistent with stiffened and unstiffened caisson installation in normal clay (Westgate et al., 2009; Gaudin et al., 2014).

b) However, for jacking installation, the caisson final penetration depth (22.85~23.20 m compared to 22.4~22.50) is slightly higher or in another word the soil heave inside the caisson (0.8~1.15 m vs. 1.5~1.6 m) is greater for jacking + suction installation. Again, this is consistent with the results in normal clay (Zhou & Randolph, 2006; Westgate et al., 2009).

c) In this calcareous silt with undrained shear strength $s_u = 1.5 + 1.7z$ kPa, the caisson maximum installation resistance narrowly ranges from 15.5 to 16.17 MN. The
equivalent range for the required underpressure is 321~335 kPa.

### 3.2 Installation Mechanism

The degree of soil backflow into the gaps between the embedded stiffeners and trapping of softer material at the base of the bottom stiffener during stiffened caisson penetration has implications for the resistance acting on the caisson during installation. In the current testing, the opacity of natural soil prevents measurement of the soil motion during penetration within the body of the soil sample. However, Zhou et al. (2016) reported results from an extensive investigation carried out through large deformation finite element analysis, varying the relevant range of various parameters related to the caisson geometry and soil strength. Three conclusions relevant to the current paper can be extracted as: first, all gaps between the embedded stiffeners below a critical depth $H_c$ will be filled by infill soil, with $H_c$ can be calculated according to (Zhou et al., 2016)

$$
\frac{H_c}{D} = \left(13.88 \frac{b}{D} + 1.02\right) \left(\frac{s_{uhc}}{\gamma D}\right)^{0.84}
$$

(2)

where $s_{uhc}$ is the shear strength at $H_c$.

Second, the soil movement is mostly found to be restricted towards inside of the caisson, with the soil outside the caisson remains more or less undisturbed. The strength inside the caisson is degraded as the soil is sheared. The softer soils trapped at the base of the bottom stiffener are from the upper layers (starting from the mudline). Third, the infill soils in the gaps between stiffeners move downward with the advancing caisson. Thus, there is no shearing between the infill soil and the inner skirt wall, but between the infill and adjacent soils. The undrained shear strength mobilised along the soil-soil sliding plane is much lower than the intact strength of the soil. No end bearing is mobilised at the base of any individual stiffeners above the bottom one. In this study, post-test inspection confirmed that (a) softer soil was
trapped at the base of the bottom stiffener, and (b) soil flowed back in the gaps between the embedded stiffeners.

These findings lead to a simple mechanism, as illustrated in Figure 5, for caisson installation in soil with low mudline strength intercept.

### 3.3 Theoretical Solutions

Conventionally used shear resistance method and piezocone based direct design approach are discussed below. The centrifuge test results will be calibrated against these design methods with the aim of back calculating the range of strain rate parameter, \( \beta \), and reduction factor for sleeve friction, \( \alpha_c \), respectively.

**Shear resistance method**

For assessing caisson penetration resistance, a shear resistance method is commonly adopted (e.g. Andersen & Jostad, 1999; Houlsby & Byrne, 2005), with variations on the inclusion and formulation of the various resistance forces. Adopting a similar approach, and following Figure 5, an expression can be proposed here as

\[
F_p = R_{it} (F_{fo} + F_{fib} + F_{fit}) + R_{fe} (F_{bt} + F_{bb}) \\
= R_{it} (\alpha_{fo} \bar{s}_{u,o} A_{fo} + \alpha_{ib} \bar{s}_{u,ib} A_{fib} + \alpha_{it} \bar{s}_{u,itt} A_{fit}) + \gamma d \left( N_{c,it} s_{u,itt} A_{bt} + \gamma' d_{b} A_{bb} + \gamma d_{b} A_{bb} \right)
\]

The frictional resistance is comprised outer caisson wall-soil friction (\( F_{fo} \)), inner caisson wall-soil friction below the bottom stiffener (\( F_{fib} \)) and inner soil-soil (neglecting faces of the embedded stiffeners) friction above the bottom stiffener (\( F_{fit} \)), whereas the bearing resistance is comprised of end bearing at the base of the skirt (\( F_{bs} \)) and bottom stiffener (\( F_{bb} \)) (Figure 5).

\( \bar{s}_{u,o} \) is the average undrained shear strength over the caisson tip penetration depth (\( d_t \)), and \( \alpha_{fo} \) and \( A_{fo} \) are the corresponding friction factor and surface area. Similarly, \( \bar{s}_{u,ib} \) and \( \bar{s}_{u,itt} \) are the
average undrained shear strength over inner caisson wall below the bottom stiffener and over
the bottom stiffener base penetration depth (d_b) respectively. N_c,t is the bearing capacity factor
of caisson tip, and s_u,0t and A_bt are the corresponding local undrained shear strength at the
caisson tip level and bearing area respectively. Similarly, s_u,0b and N_c,b are the undrained shear
strength and bearing capacity factor of the bottom stiffener.

Natural fine grained soils exhibit strain-rate dependency and also soften as they are sheared
and remoulded. The term R_f (representing both R_{ff} and R_{fe}) is therefore introduced in
Equation 3. The strain rate dependency is typically modelled using either semi-logarithmic,
power or inverse hyperbolic sine function, with the power function adopted here. The
softening is accounted for using the Einav & Randolph (2005) model, with parameter R_f
expressed as

\[
R_f = \left( \frac{\dot{\gamma}}{\dot{\gamma}_{ref}} \right)^n \left[ \delta_{rem} + (1 - \delta_{rem}) e^{-3 \delta_{rem}} \right]
\]  

(4)

where \( \dot{\gamma} \) is the operative shear strain rate, and \( \dot{\gamma}_{ref} \) is the reference strain rate at which the
undrained shear strength was measured. The first bracketed term has to be \( \geq 1 \).

During caisson penetration, the operative shear strain rate varies through the soil body, but it
is reasonable to assume that at any given location the operational strain rate may be
approximated by the normalised velocity, \( v/(t + b) \). Equation 4 can therefore be expressed as

\[
R_f = \left( \frac{v}{(t + b) \dot{\gamma}_{ref}} \right)^n \left[ \delta_{rem} + (1 - \delta_{rem}) e^{-3 \delta_{rem}} \right]
\]  

(5)

n is introduced to account for the greater rate effects for caisson surface resistance compared
to tip resistance. This is due to the higher strain rate at the cylindrical surface involving
curved shear bands, which can be estimated to be 20–40 times $v/D$ using a rigorous energy approach maintaining equilibrium (Einav & Randolph, 2006). Therefore, $n$ is taken as 1 for $F_{b_{t}}$ and $F_{b_{b}}$, and following the expression below for $F_{f_{o}}, F_{f_{b}}$ and $F_{f_{f}}$ (Zhu & Randolph, 2011; Chow et al., 2014)

$$n = 2\left(\frac{n_{1}}{\beta} + n_{1} - 2\right)$$

(6)

with $n_{1} = 1$ for axial loading.

Although the shear strength was measured using T-bar or cone penetration tests, involving $v/D$ of 0.12 to 0.2 $s^{-1}$, the resistances are also affected by strain softening, so that T-bar and piezocone bearing capacity factors of 10.5 or 13.56 used for deducing the shear strength are essentially consistent with a laboratory strain rate of $\sim 1 \times 10^{-5} /s$ (e.g. Zhou & Randolph, 2009). As such, $\dot{\gamma}_{\text{ref}}$ was taken as $\sim 1 \times 10^{-5}/s$.

The caisson penetration resistance profiles were calculated using Equations 3–6 and are shown in Figure 4. Due to geometrical similarity between the stiffener and the (half) T-bar penetrometer, $N_{c,b} = 10.5$ was selected, whereas the caisson tip was considered analogous to deeply embedded strip footings and as such $N_{c,t} = 7.5$ was taken (Andersen & Jostad, 1999; Gaudin et al., 2014). $\delta_{\text{rem}} = \sim 0.21$ (from Figure 2b) and for this quasi-static penetration, $\xi = \sim 1.8$ and $\xi_{95} = \sim 15$ (Erbrich, 2005) were considered. For caisson-soil shearing and for end bearing, with less disturbance of adjacent soil and no trapping of soil from the surface layers, i.e. for calculating $\bar{s}_{u,o}, \bar{s}_{u,b}$ and $s_{u,0t}$ undisturbed soil strength ($s_{u,p}$) was used. For soil-soil shearing and for end bearing, with significant disturbance of adjacent soil due to soil flow and dragging of softer soil from the surface layers, i.e. for calculating $\bar{s}_{u,h}$ and $s_{u,0b}$ soil strength deduced from T-bar extraction data ($s_{u,e}$) was used (see Figures 2 and 5). Equation 2 gives $H_{c}$
= 2.5 m, with the soil infill in the gaps between the embedded stiffeners were confirmed through observation of the caisson after extraction.

If the interface friction ratios $\alpha_0$, $\alpha_{ib}$, $\alpha_{it}$ are considered as equal and taken as the inverse of the soil sensitivity ($1/\sim 4.75 = 0.21$) following Andersen & Jostad (1999) and Andersen et al. (2005), the best match between Equation 3 and measured data provided rate parameter $\beta = 0.075-0.085$ (see Figure 4). The average $\beta = 0.08$ corresponds to frictional resistance that is about 40% of the total penetration resistance.

Erbrich & Hefer (2002) reported field data from installation of 9 stiffened caissons in calcareous silty clay (Laminaria field, $S_t = 3.33$). For back calculation for these data, $\delta_{rem} = \sim 0.3$; $\xi = 1.8$; $\xi_{95} = 15$; $\alpha_0 = \alpha_{ib} = \alpha_{it} = 1/S_t = 0.3$; $s_{u,p} = 10 + 1.8 z$ kPa; $s_{u,e} = 10 + 0.8 z$ kPa; $\dot{\gamma}_{ref} = 1\times 10^{-5}$ /s; $v_p = 0.0005$ m/s (Erbrich & Hefer, 2002; Erbrich, 2005) were used. The back-figured upper and lower bound profiles are included in Figure 6a, with $\beta = 0.001$ and 0.01 respectively (average total frictional resistance = 62~64% of the total penetration resistance). Gaudin et al. (2014) reported data from centrifuge model tests on a stiffened caisson installing in Gulf of Guinea clay. Calibration of Equation 3 against these data using $\delta_{rem} = \sim 0.59$; $\xi_{95} = 15$; $\alpha_0 = \alpha_{ib} = \alpha_{it} = 1/S_t = 0.59$; $s_{u,p} = 1.5 + 1.2 z$ kPa; $s_{u,e} = 0.75 + 0.9 z$ kPa; $\dot{\gamma}_{ref} = 1\times 10^{-5}$ /s; $v_p = 0.4$ mm/s (Gaudin et al., 2014) provides $\beta = 0.001$ (average total frictional resistance = $\sim 62\%$ of the total penetration resistance), as shown in Figure 6b.

**Piezocone based direct design method**

This is a direct design approach in which net tip resistance, $q_{cnet}$, and sleeve friction, $f_s$, from a piezocone penetration test (as plotted in Figure 7) are used for calculating end bearing, $F_{bt}$, $F_{bb}$, and frictional resistance, $F_{fb}$, $F_{fbb}$, $F_{fit}$, respectively. This can be expressed as

$$ F_p = \alpha \left( \bar{r}_{k,s} A_{f_c} + \bar{r}_{k,ib} A_{f_{ib}} + \bar{r}_{k,fit} A_{fit} \right) + \left( q_{cnet,t} A_{bt} + \gamma' d_A A_{bt} + q_{cnet,b} A_{bb} + \gamma' d_A A_{bb} \right) $$

(7)
where \( \bar{f}_{s,o}, \bar{f}_{s,ib}, \bar{f}_{s,sh} \) are the average sleeve friction over the caisson tip penetration depth \((d_t)\), inner caisson wall below the bottom stiffener and over the bottom stiffener base penetration depth \((d_b)\) respectively. \( q_{cnet,t} \) and \( q_{cnet,b} \) are the net tip resistance at the caisson tip and bottom stiffener base level. Expressed in equivalent prototype scale, the piezocone penetrometer diameter of 2 m is significantly greater than \( t = 0.08 \) m or \( b = 0.5 \) m and hence \( q_{cnet} \) was not averaged over a number of diameters below the piezocone shoulder. In the field, an averaging of \( q_{cnet} \) may not be necessary for end bearing at caisson wall base but at the bottom stiffener base, as the diameter of the commonly used cone penetrometer \((0.0357 \text{ m})\) is similar to caisson wall thickness of \( \leq 0.05 \) m, but relatively smaller than bottom stiffener width of 0.1~0.4 m. For the simplicity, sleeve friction from piezocone penetration test was used for both caisson-soil \((\bar{f}_{s,o}, \bar{f}_{s,ib})\) and soil-soil friction \((\bar{f}_{s,sh})\). This concept is consistent with the method proposed by Nottingham (1975) and Schmertmann (1978) for assessing pile capacity in clay, and recently adopted for back calculating torpedo anchor vertical pullout capacity in calcareous silt (Hossain et al., 2015).

Calibration of the piezocone penetration data (Figure 7) against measured installation resistance profiles provides a range of \( \alpha_c = 0.95 \) to 1.05. However, the concave predicted profile results a significant discrepancy for 4~20 m penetration depth. This can be improved by using the concept proposed by Erbrich & Hefer (2002). The undrained shear strength deduced from the T-bar penetration test data \((S_u,p, T-bar)\) was divided by a factor \((0.65)\) to match with the sleeve friction profile at deep penetration depths \( (> \sim 20 \text{ m})\) (see Figure 7), and then that profile was used for back-calculation instead of the sleeve friction profile. This approach has led to reduce the discrepancy for intermediate depths and values of \( \alpha_c \) to \( \sim 0.8 \). The back-figured curve with \( \alpha_c = 0.8 \) is shown in Figure 4. The range of back-figured \( \alpha_c = 0.8 \) to 1.05 is
within the much wider range, $\alpha_c = 0.2\sim1.25$ as reported for piles in clay (Nottingham, 1975; Schmertmann, 1978).

4 RESULTS AND DISCUSSION: CAISSON HOLDING CAPACITY

4.1 Measured Vertical Pullout Resistance

The extraction response from the caisson ‘sealed’ vertical pullout tests are presented in terms of load resistance, $F_e$, as a function of normalised pullout displacement, $d_t/D$ in Figure 8, with capacities from all tests tabulated in Table 2. The profiles in Figure 8 show a consistent trend, characterised by a sharp increase in load to an ultimate holding capacity within a displacement of $0.15\sim0.22D$, before reducing, indicating the absence of anchor rotation under this vertical pullout. Caisson holding capacity increases with increasing consolidation period $t_c$ prior to anchor pullout, reflecting the regaining of soil strength with $t_c$.

4.2 Extraction Mechanisms

It was revealed through inspection (at 1g) of the caisson after the completion of each test that a soil plug was extracted with the caisson. For vertical pullout, the plug base was somewhat conical. For lateral pullout with the mudline load inclination of 20, 40 and 80°, a significant rotation occurs during pullout, while for $\theta_0 = 0^\circ$, the caisson was moved more or less laterally along the loading direction. A camera mounted on the strongbox was allowed for monitoring the caisson movement during testing.

4.3 Theoretical Solutions for Vertical Loading

Similar to installation, two methods have been considered for this quasi-static extraction problem, including: (a) conventional shear resistance, and (b) a direct piezocone based design method.
The ‘sealed’ vertical pullout capacity of the caisson can be calculated using a reorganised version of Equation 3 that now accounts for end bearing resistance at the full base of the caisson and frictional resistance along the outer wall only according to

\[
F_e = R_{ef} (F_{fo}) + R_{fe} (F_{bf})
\]

\[
= R_{ef} (A_{bf} \alpha R_{fo}) + R_{fe} (N_{c,f} s_{u,0} A_{bf})
\]

where \(N_{c,f}\) is the bearing capacity factor of caisson full base and \(A_{bf}\) is the corresponding bearing area. It is assumed that the weight of the soil plug inside the caisson and outside of the caisson was counterbalanced. Note, the load cell readings were zeroed at the point where the caisson tip touched the soil surface, and hence the submerged weight of the anchor, \(W_{ss}\), was not taken off (i.e. measured resistance \(F_e = \text{net soil resistance } F_N\)). For this circular conical-based (with trapped soil plug), deep bearing capacity factor, \(N_{c,f} = 9.0\) was adopted for the base of the caisson, with due consideration given to the corresponding relative embedment depth of \(\sim 3D\). For this axisymmetric problem, \(\beta = 1/2\) of plane strain value of \(\sim 0.08\) (as was obtained for installation) = 0.04 was used (following Low et al., 2008). The strength mobilised during caisson pullout will be different to that mobilised during caisson installation due to significant inward flow that causes significant effective stress changes in the soil surrounding the caisson, strength regain following reconsolidation (Chen & Randolph, 2007; Hossain et al., 2015). These effects were captured by back-figuring \(\alpha_0\) from the measured holding capacity and Equation 8 using the measured (intact) shear strength profile, \(s_{u,p} = 1.5 + 1.7z\) kPa. This provides \(\alpha_0 = 0.45, 0.32, 0.21, 0.16\) and 0.05 for \(t_c = 12, 6, 3, 2\) and 0 months (Figure 8; \(\delta_{rem} = \sim 0.21; \xi_9 = 15; \dot{\gamma}_{ref} = 1 \times 10^{-5} /s; v_e = 0.4 \text{ mm/s}\), and leads to the following comments.
The upper bound $\alpha_o = 0.45$ corresponding to $t_c = 12$ months matches with $\alpha_o = 0.45$ (normally consolidated sample) reported by Randolph et al. (1998) from back analysis of data from suction caisson pullout tests after consolidation of 1.6 years in calcareous silt. However, it is significantly lower than $\alpha_o = 0.96$ that would be obtained from Equation 9 (below), which is commonly used for driven piles in clay (API, 2007)

$$\alpha_o = 0.5 \left( \frac{S_{up}}{\gamma Z} \right)^{-0.5}$$  \hspace{1cm} (9)

The trend of increasing $\alpha_o$ (or gain in friction) with increasing reconsolidation time is consistent with behaviour in Gulf of Mexico clay as illustrated in Figure 9, although the rate of gaining capacity or increasing friction factor is much slower for calcareous silt. Note, Jeanjean (2006) reported vertical pullout capacity for suction caissons anchors ($D = 2.9\sim\sim 5.5 \text{ m}$) installed at various sites (sites A, C, D, G) of the Gulf of Mexico. The seabeds consisted of lightly overconsolidated clay ($S_t = 2\sim\sim 4$) with strength increasing somewhat linearly with depth. The effect of reconsolidation period on back-figured friction factor (calculated following Equation 8) is featured in Figure 9 with $\alpha_o = 0.75\sim\sim 0.9$ (this upper bound corresponds to 90% consolidation, which is reported to be taken place in 30 days for Gulf of Mexico clays) and 0.35\sim\sim 0.45$ for $t_c = 1.7\sim\sim 45$ and 0 months respectively. For Gulf of Mexico clays, Clukey et al. (2013) also reported values of $\alpha_o = 0.58\sim\sim 0.65$ and $0.25\sim\sim 0.35$ for 90% set-up and initial (immediately after installation).

The lower bound $\alpha_o = 0.05$ corresponding to $t_c = 0$ represents short term fully remoulded friction ratio. This value is significantly lower than $\alpha_o = 1/S_t = 0.21$, but consistent with the value of 0.05 (for T-bar strength profile) for Laminaria calcareous
silty clay as commented by Erbrich & Hefer (2002) for caisson installation. However, it is significantly lower than $\alpha_o = 0.25\sim0.45$ for Gulf of Mexico clays as just noted.

(d) In contrast to installation phase, reverse end bearing resistance over the full caisson base corresponds to 84% of the pullout capacity for $t_c = 12$ months, which increases to 95% for $t_c = 0$ month.

(e) The loss of suction resulted in a $\sim50\%$ loss of vertical pullout capacity (Vented tests T4 and T9 compared to sealed test T3; Table 2 and Figure 8). Analysing the data from caisson installation and retrieval in the Gulf of Mexico clays, Clukey et al. (2013) commented that the corresponding loss may be significant.

**Piezocone based direct design method**

Net tip resistance, $q_{cnet}$, and sleeve friction, $f_s$, from a piezocone extraction test (carried out with $t_c = 12$ months; see Figure 7) are used for calculating end bearing, $F_{bf}$, and frictional resistance, $F_f$, respectively. This can be expressed as

$$F_e = \alpha_s \left( \bar{f}_{s,0} A_{f_0} \right) + \left( \bar{q}_{cnet,1} A_{bf} \right)$$

The piezocone penetrometer diameter of 2 m is significantly smaller than $D = 8$ m and hence $q_{cnet}$ was averaged over 0.2D below and 0.4D above the caisson base, being consistent with the design of pile and spudcan foundations (Schmertmann, 1978; InSafeJIP, 2011). Calibration of the piezocone extraction data (Figure 7) against measured pullout resistance profile for $t_c = 12$ months provides $\alpha_c = \sim0.2$. The calculated profile is included in Figure 8.

**4.4 Effect of Mudline Load Inclination, $\theta_0$**

In the above sections, pullout has been considered as vertical with the extraction carried out by means of a shaft linked between the caisson top and the actuator. The angle to the horizontal at the mudline was $\theta_0 = 90^\circ$ (= the angle at the anchor top). However, it may vary
with the mooring system e.g. $\theta_0 = 0\sim15^\circ$ for catenary mooring and $\theta_0 = 35\sim55^\circ$ for taut leg mooring. To investigate the effect of mudline load inclination, four tests were therefore carried out at $\theta_0 = 0, 20, 40$ and $80^\circ$ (Tests T10~T13; Table 2), with the extraction executed using a mooring chain connected to the padeye (0.7L below the lid). For all these tests $t_e$ was identical of 12 months. The extraction resistance profiles are shown in Figure 10a, from which it is evident that (a) the distance of the mooring line cutting through the soil was longer for lower $\theta_0$, and (b) the degree of rotation required for the anchor to be aligned with the pulling line decreases or the brittleness of failure increases with decreasing $\theta_0$. For vertical pullouts with $\theta_0 = 90^\circ$ (from the caisson top), soil disturbance was limited to a small zone in the immediate vicinity of the anchor. In contrast, for $\theta_0 < 90^\circ$, the caisson rotated and underwent significant displacement in the direction of the load application, disturbing a much larger zone along the anchor trajectory (both loading and the trailing direction).

The holding capacities for all tests are tabulated in Table 2. It is seen that, for vertical pullout, anchoring capacity is 2.3~2.8 times the dry caisson weight ($W_d$). For $\theta_0 = 0$, this increased to $\sim4.8W_d$.

The net capacity $F_N (= F_e - W_{ss} - W_p)$ was divided into horizontal, $F_{N,H}$, and vertical, $F_{N,V}$, components according to the loading angle at the padeye, $\theta_a$, which was calculated using the Neubecker & Randolph (1995) approach ($\theta_0 = 0, 20, 40, 80^\circ$ corresponds to $\theta_a = 15, 32, 40, 80^\circ$ respectively; for $\theta_0 \geq 40^\circ$, $\theta_0 = \theta_a$ owing to the location of the chain-pulley system). For extraction with the mooring line, the load cell readings were zeroed at the beginning of pullout, necessitating to negate $W_{ss}$ (given in Table 1) in addition to the weight of the soil plug trapped inside the caisson ($W_p$; calculated using $\gamma' = 4.5$ kN/m$^3$ for the plug soil). Figure 10b shows ultimate limit states in terms of loads, indicating the size of the failure envelope in $H$ (horizontal)-$V$ (vertical) space. Following Supachawarote (2007), it is assumed that the net
capacity for $\theta_a = 90^\circ$ is consistent to the vertical component $F_{N,V}$ for $\theta_a = 80^\circ$, and the net
capacity for $\theta_a = 0^\circ$ is about 2.6% higher than that for $\theta_a = 15^\circ$. These data are also included
in Figure 10b, allowing better definition of the full envelope (see Figures 10b and 10c).
Figure 10c represents the maximum states normalised by the maximum loads, $F_{N,V} =
F_{N,V}/F_{N,V_{\text{max}}}$ and $F_{N,h} = F_{N,H}/F_{N,H_{\text{max}}}$, indicating the shape and relative size of the failure
envelope.
Randolph & House (2002) reported a failure envelope for a suction caisson (with length to
diameter ratio of $\sim 6$ and load applied at the padeye) in clay. A power law can be proposed to
describe the shape of the normalised failure envelope as

$$F_{N,v} = \left(1 - F_{N,h}^q\right)^p$$  \hspace{1cm} (11)

with $p = 3$ and $q = 5$. Failure envelopes, established using results from finite element analyses,
were proposed for caissons (load applied at the padeye) by Cho & Bang (2002),
Supachawarote (2007), with $p = 4.5-L/3D = 3.5$ and $q = 0.5+L/D = 3.5$, and Ahn et al. (2013),
with $p = 2.82$ and $q = 2.84$. These envelopes are included in Figure 10c, where it can be seen
that the best agreement with the measured data are with the envelope proposed by

### 4.5 Effect of Reconsolidation Time, $t_c$ after Caisson Installation

Caisson installation induces excess pore water pressures that initially reduce the available soil
strength. After installation the excess pore pressures dissipate, causing a regain in the strength
of the soil surrounding the caisson. Figure 8 shows the results from three tests involving pure
vertical loading with reconsolidation times $t_c = 0, 2, 3, 6$ and 12 months ($\theta_0 = 90^\circ$, Tests
T1~T3 and T5~T8; Table 2). The coefficient of friction, $\alpha$, was shown previously to increase
with $t_c$ and as expected this increase is also reflected in an increase in the caisson capacity.
The rate at which reconsolidation surrounding a cylindrical caisson takes place can be approximated from the dissipation phase of the piezocone penetrometer test (see Figure 2c), noting that the rate of consolidation is linked to $D^2$, and the ratio $D^2$ between the piezocone and the caisson is 0.063. It can be seen from Figure 2c that around 43% and 56% consolidation took place at $t_c = 6$ and 12 months respectively.

Following the approach adopted by Jeanjean (2006), Richardson et al. (2009) and Hossain et al. (2015), the relative increase in anchor capacity was determined through consideration of the net capacity $F_N$ relative to the immediate capacity $F_{N,0}$ and the ultimate long term capacity $F_{N,max}$, and assumed linked to the degree of consolidation by

$$\frac{F_N - F_{N,0}}{F_{N,max} - F_{N,0}} \approx 1 - \frac{\Delta u}{\Delta u_{max}}$$  \hspace{1cm} (12)

Figure 11 shows the results (with anchor capacity regain calculated using Equation 12) for Tests T1~T3 and T5~T8, plotted against dimensionless time, $T = \frac{cht_c}{D^2}$, indicating the progression of caisson capacity (due to reconsolidation) with time after installation.

**Theoretical solution**

Richardson et al. (2009) showed that the gain in anchor capacity with reconsolidation time can be modelled using the cavity expansion method (CEM) for radial consolidation (Randolph & Wroth, 1979) following creation of a cylindrical cavity (simulating installation of a solid, close-ended pile). Figure 11 includes the degree of consolidation predicted by the CEM for $I_r = 150$ (as derived from simple shear test data). The theoretical solution provides a reasonable representation of the measured increase in capacity with time for the caisson and indicates that for the caisson and calcareous silt considered here ($D = 8$ m), approximately 27% of the long term anchor capacity would be available within 1 year of caisson installation.
1.07 m; Hossain et al., 2015) is also accurately estimated by the theoretical solution. The field data of suction caissons (D = 2.9~5.5) installed in the Gulf of Mexico clay reported by Jeanjean (2006) are scattered showing no particular trend.

5 CONCLUDING REMARKS

This paper has reported results from centrifuge model tests investigating installation and monotonic pullout of a suction caisson anchor in calcareous silt. For assessing underpressure required for installation of a suction caisson, a conventional shear resistance model (Equation 3), accounting for strain softening and rate dependent undrained shear strength, was shown to be capable of predicting measured penetration resistance profiles with friction factor for outer and inner surfaces of 0.21 and rate parameter of 0.075~0.085. An alternative piezocone based direct design approach (Equation 7), deriving caisson end bearing and frictional resistance from piezocone tip resistance and sleeve friction respectively, was also proposed and was seen to be capable of modelling the caisson penetration response using a reduction factor on sleeve friction of ~0.8.

Caisson holding capacity under pure vertical loading (load applied at the top of the caisson), was shown to be well described using a shear resistance model based on conventional frictional and (reverse) end bearing resistance (Equation 8) using a friction factor of 0.45 (for reconsolidation period of 12 months) and rate parameter of 0.04. The value of friction factor (and hence contribution from frictional resistance) was shown to be reduced with decreasing reconsolidation period. The piezocone-based direct design approach (Equation 10) was seen to be capable of predicting the caisson maximum extraction resistance using a reduction factor on sleeve friction of ~0.2.

Caisson holding capacity under inclined monotonic loading was presented as a combined vertical and horizontal loading failure envelope, which was well represented by finite element
and limit analysis based envelopes developed for embedded caissons in clay (Equation 11).

The regain of caisson capacity due to reconsolidation of the soil surrounding the embedded anchor was shown to agree well with a cavity expansion based theoretical prediction.

6 ACKNOWLEDGEMENTS

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NOTATION

Abb  bottom stiffener base area

Abf  caisson full base area

Abt  caisson tip area

A\textsubscript{fib}  caisson inner surface area below bottom stiffener base

A\textsubscript{fit}  caisson inner surface area above bottom stiffener along faces of stiffeners

Afo  caisson outer surface area

b  ring stiffener width

ch  coefficient of horizontal consolidation

cv  coefficient of vertical consolidation

D  suction caisson diameter

Dep  object projected area equivalent diameter

db  bottom stiffener base penetration depth

dt  caisson tip penetration depth

F\textsubscript{bb}  end bearing resistance at bottom stiffener base

F\textsubscript{bf}  end bearing resistance at caisson full base

F\textsubscript{bt}  end bearing resistance at caisson tip

F\textsubscript{e}  extraction resistance

F\textsubscript{e,\text{max}}  maximum extraction resistance

F\textsubscript{fib}  inner caisson wall-soil friction below bottom stiffener base

F\textsubscript{fit}  inner soil-soil friction above bottom stiffener base along faces of stiffeners
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s           ring stiffener spacing (centre to centre)

$S_u$       undrained shear strength

$S_{um}$    undrained shear strength at mudline

$S_{u,e}$   undrained shear strength from penetrometer extraction test

$S_{u,p}$   undrained shear strength from penetrometer penetration test

$S_{u,p,T-bar}$ undrained shear strength from T-bar penetration test

$S_{u,0b}$  local undrained shear strength at bottom of stiffener base level

$S_{u,0t}$  local undrained shear strength at caisson tip level

$S_{u,H_c}$ undrained shear strength at cavity base level i.e. at $H_c$

$S_{o,o}$   average undrained shear strength over caisson tip penetration depth

$S_{o,ib}$  average undrained shear strength over caisson tip and below bottom stiffener base

$S_{o,it}$  average undrained shear strength over bottom stiffener base penetration depth

$T$ and $T^*$ dimensionless time

$t$          caisson wall thickness

$t_c$       reconsolidation time

$t_d$       dissipation time

$U$         normalised excess pore pressure

$V$         dimensionless velocity

$v$         object penetrating velocity

$ve$        caisson penetrating velocity
| 673 | $v_p$ | caisson extracting velocity |
| 674 | $W_d$ | caisson dry weight |
| 675 | $W_p$ | weight of soil plug trapped inside caisson |
| 676 | $W_s$ | caisson submerged weight in water |
| 677 | $W_{ss}$ | caisson submerged weight in soil |
| 678 | $w$ | distance of bottom stiffener base from caisson tip |
| 679 | $z$ | depth below soil surface |
| 680 | $\alpha_c$ | reduction factor for sleeve friction |
| 681 | $\alpha_0$ | friction factor for caisson outer wall-soil interface |
| 682 | $\alpha_{ib}$ | friction factor for caisson inner wall-soil interface below bottom stiffener |
| 683 | $\alpha_{it}$ | friction factor for caisson inner soil-soil interface along faces of stiffeners |
| 684 | $\beta$ | rate parameter for power expression |
| 685 | $\Delta u$ | excess pore pressure |
| 686 | $\Delta u_{\text{max}}$ | maximum excess pore pressure |
| 687 | $\delta_{\text{rem}}$ | remoulded strength ratio |
| 688 | $\dot{\gamma}$ | shear strain rate |
| 689 | $\dot{\gamma}_{\text{ref}}$ | reference shear strain rate |
| 690 | $\gamma'$ | effective unit weight of soil |
| 691 | $\theta_a$ | padeye load inclination |
| 692 | $\theta_0$ | mudline load inclination |
693 \( \xi \)  cumulative plastic shear strain

694 \( \xi_{95} \)  cumulative plastic shear strain required for 95% remoulding


Table 1. Model and prototype caisson dimensions

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</tr>
<tr>
<td>Thickness ratio</td>
<td>D/t</td>
<td>100.00</td>
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<tr>
<td>Ring stiffener width</td>
<td>b</td>
<td>2.50 mm</td>
<td>0.50 m</td>
</tr>
<tr>
<td>Ring stiffener height</td>
<td>h</td>
<td>1.00 mm</td>
<td>0.20 m</td>
</tr>
<tr>
<td>Ring stiffener spacing (c/c)</td>
<td>s</td>
<td>20.00 mm</td>
<td>4.00 m</td>
</tr>
<tr>
<td>Distance of bottom stiffener base from</td>
<td>w</td>
<td>19.50 mm</td>
<td>3.90 m</td>
</tr>
<tr>
<td>caisson tip</td>
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<td></td>
<td></td>
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<tr>
<td>Dry weight</td>
<td>W_d</td>
<td>1.50 N</td>
<td>12.01 MN</td>
</tr>
<tr>
<td>Submerged weight in water</td>
<td>W_s</td>
<td>1.00 N</td>
<td>8.05 MN</td>
</tr>
<tr>
<td>Submerged weight in soil</td>
<td>W_ss</td>
<td>0.68 N</td>
<td>5.40 MN</td>
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### Table 2. Summary of centrifuge tests conducted

<table>
<thead>
<tr>
<th>Test</th>
<th>Method</th>
<th>Installation depth: m</th>
<th>Maximum installation resistance, $F_{p,\text{max}}$: MN</th>
<th>Model: min</th>
<th>Prototype: month</th>
<th>Extraction mode</th>
<th>Mudline load inclination, $\theta_o$: $^\circ$</th>
<th>Padeye load inclination, $\theta_o$: $^\circ$</th>
<th>Holding capacity</th>
</tr>
</thead>
<tbody>
<tr>
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<td>$F_{e,\text{max}}$: MN</td>
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<tr>
<td>T1</td>
<td>Jacking + suction</td>
<td>22.40</td>
<td>15.50</td>
<td>13.14</td>
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<td>90</td>
<td>90</td>
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<td>T3</td>
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<td>15.60</td>
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<td>T4</td>
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<td>22.40</td>
<td>15.55</td>
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<td>T5</td>
<td>Jacking</td>
<td>23.20</td>
<td>16.17</td>
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<td>T9</td>
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<td>23.00</td>
<td>16.15</td>
<td>2.16</td>
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<td>Vented</td>
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<td>T10</td>
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<td>16.01</td>
<td>13.14</td>
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<td>T11</td>
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<td>22.90</td>
<td>16.07</td>
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<td>T12</td>
<td></td>
<td>23.00</td>
<td>16.12</td>
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</tr>
</tbody>
</table>

Installation rate, $v_p = \text{extraction rate, } v_e = 0.4 \text{ mm/s}$
Number of Figure: 11

Figure 1. Suction caisson anchor: (a) Schematic representation of installed suction caisson anchor and idealised seabed strength profile (PPT: pore pressure transducer; TPT: total pressure transducer); (b) Model stiffened caisson

Figure 2. Results from soil characterisation tests using T-bar and piezocone: (a) Undrained shear strength profiles; (b) Undrained shear strength degradation during T-bar cyclic sequence; (c) Pore water pressure ($u_2$ position) during piezocone test; (d) Interpretation of $c_t$ from piezocone dissipation data

Figure 3. Schematic representation of experimental arrangement for caisson installation and pullout

Figure 4. Installation resistance profiles and theoretical upper and lower bound predictions

Figure 5. Simplified installation mechanism following Zhou et al. (2016) (see also Equations 2 and 3)

Figure 6. Theoretical predictions for existing data: (a) Field data of stiffened caisson installation in calcareous silty clay (Erbrich & Hefer, 2002) and theoretical upper and lower bound predictions; (b) Centrifuge test data of a stiffened caisson installation in Gulf of Guinea clay (Gaudin et al., 2014) and theoretical prediction

Figure 7. Profiles of net tip resistance, $q_{net}$, and sleeve friction, $f_s$, from piezocone test

Figure 8. Vertical extraction resistance profile ($\theta_a = 0^0 = 90^0$, pullout from caisson top; Tests T1~T9, Table 2)
Figure 9. Effect of reconsolidation time on friction factor, $\alpha_0$, back-figured from measured holding capacity

Figure 10. Caisson capacity under inclined loading: (a) Effect of mudline load inclination $\theta_0$ on extraction resistance (Tests T10–T13; Table 2); (b) Net holding capacity under vertical and horizontal loading; (c) Failure envelope for vertical and horizontal loading in normalised load space

Figure 11. Dependence of reconsolidation time after installation on caisson capacity
(a) Schematic representation of installed suction caisson anchor and idealised seabed strength profile (PPT: pore pressure transducer; TPT: total pressure transducer)
(b) Model stiffened caisson

Figure 1. Suction caisson anchor
Undrained shear strength, $s_u$: kPa

$s_{u,e} (t_c = 0) = 0.5 + 1.1z$ kPa

$s_{u,e} (t_c = 12 \text{ months}) = 0.5 + 1.5z$ kPa

$s_{u,p} = 1.5 + 1.7z$ kPa

(a) Undrained shear strength profiles
(b) Undrained shear strength degradation during T-bar cyclic sequence
(c) Pore water pressure ($u_2$ position) during piezocone test
Theoretical solution: Equation 1
(Teh & Houlsby, 1991)

This study: at 15.5 m
\( I_r = 150, c_h = 7 \text{ m}^2/\text{year} \)

This study: at 22.5 m
\( I_r = 150, c_h = 14.5 \text{ m}^2/\text{year} \)

(d) Interpretation of \( c_h \) from piezocone dissipation data

Figure 2. Results from soil characterisation tests using T-bar and piezocone
Figure 3. Schematic representation of experimental arrangement for caisson installation and pullout.
Figure 4. Installation resistance profiles and theoretical upper and lower bound predictions

Theoretical prediction (Eqn 3):
\[ \alpha_o = \alpha_{ib} = \alpha_{it} = 0.21; \ H_c = 2.5 \ m \]
\[ \beta = 0.075, 0.085 \]

Peizocone based prediction (Eqn 7):
\[ \alpha_c = 0.8 \]
Figure 5. Simplified installation mechanism following Zhou et al. (2016) (see also Equations 2 and 3)
Theoretical prediction (Eqn 3):
\[ \alpha_o = \alpha_{ib} = \alpha_{it} = 0.30; H_c = 5.8 \text{ m} \]
\[ \beta = 0.001, 0.01 \]

Field data:
Calcareous silty clay

6(a) Field data of stiffened caisson installation in calcareous silty clay (Erbrich & Hefer, 2002) and theoretical upper and lower bound predictions
Centrifuge test data: High plasticity clay

Theoretical prediction (Eqn 3):
\[ \alpha_o = \alpha_{ib} = \alpha_{it} = 0.59; \ H_c = 4.6 \ m \]
\[ \hat{\beta} = 0.001 \]

Figure 6. Theoretical predictions for existing data

6(b) Centrifuge test data of a stiffened caisson installation in Gulf of Guinea clay (Gaudin et al., 2014) and theoretical prediction
Figure 7. Profiles of net tip resistance, $q_{cnet}$, and sleeve friction, $f_s$, from piezocone test.
Tests T1~T9, Table 2)

Figure 8. Vertical extraction resistance profile ($\theta_a = \theta_0 = 90^\circ$, pullout from caisson top;
Figure 9. Effect of reconsolidation time on friction factor, $\alpha_o$, back-figured from measured holding capacity.

Gulf of Mexico clay:
- $S_t = 2.0\text{--}4.0$
- $c_v$ not given
  (Jeanjean, 2006)

Calcareous silt:
- $S_t = 4.5\text{--}5.0$
- $c_v = \sim1.2\text{ m}^2\text{/y}$
  (This study)
Tests T10, T11, T12 and T13:
θ₀ = 80, 40, 20, 0°
θₐ = 80, 40, 32, 15°

(a) Effect of mudline load inclination θ₀ on extraction resistance (Tests T10–T13; Table 2)
(b) Net holding capacity under vertical and horizontal loading
Figure 10. Caisson capacity under inclined loading

(c) Failure envelope for vertical and horizontal loading in normalised load space
Figure 11. Dependence of reconsolidation time after installation on caisson capacity