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

Kai Xiang Koh¹, Muhammad Shazzad Hossain² and Youngho Kim³

¹Research Student, Centre for Offshore Foundation Systems (COFS), The University of Western Australia, Tel: +61 8 6488 3974, Fax: +61 8 6488 1044, Email: kaixiang.koh@research.uwa.edu.au

²Corresponding Author, Associate Professor (BEng, MEng, PhD, MIEAust), ARC DECRA Fellow, Centre for Offshore Foundation Systems (COFS), The University of Western Australia, 35 Stirling highway, Crawley, WA 6009, Tel: +61 8 6488 7358, Fax: +61 8 6488 1044, Email: muhammad.hossain@uwa.edu.au

³Research Associate (PhD), Centre for Offshore Foundation Systems (COFS), The University of Western Australia, Tel: +61 8 6488 7725, Fax: +61 8 6488 1044, Email: youngho.kim@uwa.edu.au

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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

1 INTRODUCTION

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 (θ_0) are respectively $10^\circ \sim 25^\circ$ and $30^\circ \sim 60^\circ$ 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 ($L/D = 0.4\sim 0.5$) 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) $\times$ 390 (width) $\times$ 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 mm $\times$ 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 $N_{\text{T-bar}} = 10.5$ was used for the measured T-bar resistance and a constant bearing factor of $N_{\text{kt}} = 13.56$ (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 $s_{u,e} = 0.5 + 1.1z \text{ kPa}$ for $t_c = 0$ month and $s_{u,e} = 0.5 + 1.5z \text{ kPa}$ 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 ± 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

$$U = \frac{\Delta u}{\Delta u_{\max}} = (0.85 + 10T^*)^{-0.45} - 0.08 \quad \text{where } T^* = \frac{c_h t_d}{\sqrt{I_r R_c^2}} \quad (1)$$

where U is the normalised excess pore pressure and the rigidity index I_r of ~ 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 $v = 0.4$ 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 $v = 0.4$ 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, $V = vD/c_v \sim 420 > 30$, which was sufficient to ensure undrained conditions. For vented extraction (Tests T4 and T9; Table 2), $V = v(t + b)/c_v \sim 30.5 > 30$ 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)

$$\frac{H_c}{D} = \left(13.88 \frac{b}{D} + 1.02 \right) \left(\frac{s_{uH_c}}{\gamma' D} \right)^{0.84} \quad (2)$$

where s_{uH_c} 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, β , and reduction factor for sleeve friction, α_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

$$F_p = R_{ff}(F_{fo} + F_{fib} + F_{fit}) + R_{fe}(F_{bt} + F_{bb})$$

$$= R_{ff}(\alpha_o \bar{s}_{u,o} A_{fo} + \alpha_{ib} \bar{s}_{u,ib} A_{fib} + \alpha_{it} \bar{s}_{u,it} A_{fit}) + R_{fe}(N_{c,t} s_{u,0t} A_{bt} + \gamma' d_t A_{bt} + N_{c,b} s_{u,0b} A_{bb} + \gamma' d_b A_{bb})$$
(3)

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_{bt}) 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 α_o and A_{fo} are the corresponding friction factor and surface area. Similarly, $\bar{s}_{u,ib}$ and $\bar{s}_{u,it}$ 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

$$R_f = \left(\frac{\dot{\gamma}}{\dot{\gamma}_{ref}} \right)^\beta \left[\delta_{rem} + (1 - \delta_{rem}) e^{-3\zeta/\xi_{95}} \right] \quad (4)$$

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 ≥ 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

$$R_f = \left(n \frac{\left(\frac{v}{t + b} \right)}{\dot{\gamma}_{ref}} \right)^\beta \left[\delta_{rem} + (1 - \delta_{rem}) e^{-3\zeta/\xi_{95}} \right] \quad (5)$$

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_{bt} and F_{bb} , and following the expression below for F_{fo} , F_{fib} and F_{fit} (Zhu & Randolph, 2011; Chow et al., 2014)

$$n = 2 \left(\frac{n_1}{\beta} + n_1 - 2 \right) \quad (6)$$

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, $\dot{\gamma}_{ref}$ 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). $\delta_{rem} = \sim 0.21$ (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 $\bar{s}_{u,o}$, $\bar{s}_{u,ib}$ 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 $\bar{s}_{u,it}$ 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 α_0 , α_{ib} , α_{it} are considered as equal and taken as the inverse of the soil sensitivity ($1/4.75 = 0.21$) following Andersen & Jostad (1999) and Andersen et al. (2005), the best match between Equation 3 and measured data provided rate parameter $\beta = 0.075\sim 0.085$ (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, $\delta_{rem} = \sim 0.3$; $\xi = 1.8$; $\xi_{95} = 15$; $\alpha_0 = \alpha_{ib} = \alpha_{it} = 1/S_t = 0.3$; $s_{u,p} = 10 + 1.8z$ kPa; $s_{u,e} = 10 + 0.8z$ 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 $\delta_{rem} = \sim 0.59$; $\xi_{95} = 15$; $\alpha_0 = \alpha_{ib} = \alpha_{it} = 1/S_t = 0.59$; $s_{u,p} = 1.5 + 1.2z$ kPa; $s_{u,e} = 0.75 + 0.9z$ 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 = ~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_{f0} , F_{fib} , F_{fit} , respectively. This can be expressed as

$$F_p = \alpha_c (\bar{f}_{s,o} A_{fo} + \bar{f}_{s,ib} A_{fib} + \bar{f}_{s,it} A_{fit}) + (q_{cnet,t} A_{bt} + \gamma' d_t A_{bt} + q_{cnet,b} A_{bb} + \gamma' d_b A_{bb}) \quad (7)$$

370 where $\bar{f}_{s,o}$, $\bar{f}_{s,ib}$, $\bar{f}_{s,it}$ are the average sleeve friction over the caisson tip penetration depth (d_t),
 371 inner caisson wall below the bottom stiffener and over the bottom stiffener base penetration
 372 depth (d_b) respectively. $q_{cnet,t}$ and $q_{cnet,b}$ are the net tip resistance at the caisson tip and bottom
 373 stiffener base level. Expressed in equivalent prototype scale, the piezocone penetrometer
 374 diameter of 2 m is significantly greater than $t = 0.08$ m or $b = 0.5$ m and hence q_{cnet} was not
 375 averaged over a number of diameters below the piezocone shoulder. In the field, an averaging
 376 of q_{cnet} may not be necessary for end bearing at caisson wall base but at the bottom stiffener
 377 base, as the diameter of the commonly used cone penetrometer (0.0357 m) is similar to
 378 caisson wall thickness of ≤ 0.05 m, but relatively smaller than bottom stiffener width of
 379 0.1~0.4 m. For the simplicity, sleeve friction from piezocone penetration test was used for
 380 both caisson-soil ($\bar{f}_{s,o}$, $\bar{f}_{s,ib}$) and soil-soil friction ($\bar{f}_{s,it}$). This concept is consistent with the
 381 method proposed by Nottingham (1975) and Schmertmann (1978) for assessing pile capacity
 382 in clay, and recently adopted for back calculating torpedo anchor vertical pullout capacity in
 383 calcareous silt (Hossain et al., 2015).

384 Calibration of the piezocone penetration data (Figure 7) against measured installation
 385 resistance profiles provides a range of $\alpha_c = 0.95$ to 1.05. However, the concave predicted
 386 profile results a significant discrepancy for 4~20 m penetration depth. This can be improved
 387 by using the concept proposed by Erbrich & Hefer (2002). The undrained shear strength
 388 deduced from the T-bar penetration test data ($S_{u,p}$, T-bar) was divided by a factor (0.65) to match
 389 with the sleeve friction profile at deep penetration depths ($> \sim 20$ m) (see Figure 7), and then
 390 that profile was used for back-calculation instead of the sleeve friction profile. This approach
 391 has led to reduce the discrepancy for intermediate depths and values of α_c to ~ 0.8 . The back-
 392 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, $\alpha_c = 0.2 \sim 1.25$ 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 $0.15 \sim 0.22D$, 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.

416 ***Shear resistance method***

417 The ‘sealed’ vertical pullout capacity of the caisson can be calculated using a reorganised
 418 version of Equation 3 that now accounts for end bearing resistance at the full base of the
 419 caisson and frictional resistance along the outer wall only according to

$$\begin{aligned}
 F_e &= R_{ff}(F_{fo}) + R_{fe}(F_{bf}) \\
 &= R_{ff}(\alpha_o \bar{s}_{u,o} A_{fo}) + R_{fe}(N_{c,f} s_{u,0t} A_{bf})
 \end{aligned}
 \tag{8}$$

421 where $N_{c,f}$ is the bearing capacity factor of caisson full base and A_{bf} is the corresponding
 422 bearing area. It is assumed that the weight of the soil plug inside the caisson and outside of
 423 the caisson was counterbalanced. Note, the load cell readings were zeroed at the point where
 424 the caisson tip touched the soil surface, and hence the submerged weight of the anchor, W_{ss} ,
 425 was not taken off (i.e. measured resistance F_e = net soil resistance F_N). For this circular
 426 conical-based (with trapped soil plug), deep bearing capacity factor, $N_{c,f} = 9.0$ was adopted
 427 for the base of the caisson, with due consideration given to the corresponding relative
 428 embedment depth of $\sim 3D$. For this axisymmetric problem, $\beta = 1/2$ of plane strain value of
 429 ~ 0.08 (as was obtained for installation) = 0.04 was used (following Low et al., 2008). The
 430 strength mobilised during caisson pullout will be different to that mobilised during caisson
 431 installation due to significant inward flow that causes significant effective stress changes in
 432 the soil surrounding the caisson, strength regain following reconsolidation (Chen &
 433 Randolph, 2007; Hossain et al., 2015). These effects were captured by back-figuring α_o from
 434 the measured holding capacity and Equation 8 using the measured (intact) shear strength
 435 profile, $s_{u,p} = 1.5 + 1.7z$ kPa. This provides $\alpha_o = 0.45, 0.32, 0.21, 0.16$ and 0.05 for $t_c = 12, 6,$
 436 $3, 2$ and 0 months (Figure 8; $\delta_{rem} = \sim 0.21$; $\xi_{95} = 15$; $\dot{\gamma}_{ref} = 1 \times 10^{-5}$ /s; $v_e = 0.4$ mm/s), and leads
 437 to the following comments.

(a) 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)

$$\alpha_o = 0.5 \left(\frac{S_{u,p}}{\gamma' z} \right)^{-0.5} \quad (9)$$

(b) The trend of increasing α_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 ($D = 2.9\sim 5.5$ m) installed at various sites (sites A, C, D, G) of the Gulf of Mexico. The seabeds consisted of lightly overconsolidated clay ($S_t = 2\sim 4$) 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 $\alpha_o = 0.75\sim 0.9$ (this upper bound corresponds to 90% consolidation, which is reported to be taken place in 30 days for Gulf of Mexico clays) and $0.35\sim 0.45$ for $t_c = 1.7\sim 45$ and 0 months respectively. For Gulf of Mexico clays, Clukey et al. (2013) also reported values of $\alpha_o = 0.58\sim 0.65$ and $0.25\sim 0.35$ for 90% set-up and initial (immediately after installation).

(c) 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 $\alpha_o = 0.25\sim 0.45$ 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 $\sim 50\%$ 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_{f0} , respectively. This can be expressed as

$$F_e = \alpha_c (\bar{f}_{s,o} A_{fo}) + (\bar{q}_{cnet,t} A_{bf}) \quad (10)$$

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 $\alpha_c = \sim 0.2$. The calculated profile is included in Figure 8.

4.4 Effect of Mudline Load Inclination, θ_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. $\theta_0 = 0\sim 15^\circ$ for catenary mooring and $\theta_0 = 35\sim 55^\circ$ 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° (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_c 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 θ_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 θ_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 $\sim 4.8W_d$.

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, θ_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 $\gamma' = 4.5 \text{ kN/m}^3$ 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, $F_{N,v} = F_{N,v}/F_{N,vmax}$ and $F_{N,h} = F_{N,h}/F_{N,hmax}$, 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 ~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

$$F_{N,v}^p = (1 - F_{N,h}^q) \quad (11)$$

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 Supachawarote (2007) and Randolph & House (2002).

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, α , 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

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

Figure 11 shows the results (with anchor capacity regain calculated using Equation 12) for Tests T1~T3 and T5~T8, plotted against dimensionless time, $T = c_{ht}/D^2$, 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. Figure 11 also illustrates that regain of capacity for a torpedo anchor in calcareous silt ($D =$

1.07 m; Hossain et al., 2015) is also accurately estimated by the theoretical solution. The field data of suction caissons ($D = 2.9\sim 5.5$) 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.

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590 **NOTATION**

591	A_{bb}	bottom stiffener base area
592	A_{bf}	caisson full base area
593	A_{bt}	caisson tip area
594	A_{fib}	caisson inner surface area below bottom stiffener base
595	A_{fit}	caisson inner surface area above bottom stiffener along faces of stiffeners
596	A_{fo}	caisson outer surface area
597	b	ring stiffener width
598	c_h	coefficient of horizontal consolidation
599	c_v	coefficient of vertical consolidation
600	D	suction caisson diameter
601	D_{ep}	object projected area equivalent diameter
602	d_b	bottom stiffener base penetration depth
603	d_t	caisson tip penetration depth
604	F_{bb}	end bearing resistance at bottom stiffener base
605	F_{bf}	end bearing resistance at caisson full base
606	F_{bt}	end bearing resistance at caisson tip
607	F_e	extraction resistance
608	$F_{e,max}$	maximum extraction resistance
609	F_{fib}	inner caisson wall-soil friction below bottom stiffener base
610	F_{fit}	inner soil-soil friction above bottom stiffener base along faces of stiffeners

611	F_{fo}	outer caisson wall-soil friction
612	F_N	net capacity of caisson
613	$F_{N,0}$	immediate caisson capacity
614	$F_{N,max}$	ultimate long term caisson capacity
615	$F_{N,H}$	horizontal component of net capacity
616	$F_{N,Hmax}$	maximum horizontal component of net capacity
617	$F_{N,h}$	normalised horizontal component of net capacity
618	$F_{N,V}$	vertical component of net capacity
619	$F_{N,Vmax}$	maximum vertical component of net capacity
620	$F_{N,v}$	normalised vertical component of net capacity
621	F_p	penetration resistance
622	$F_{p,max}$	maximum penetration resistance
623	f_s	piezocone sleeve friction
624	$\bar{f}_{s,o}$	average sleeve friction over caisson tip penetration depth
625	$\bar{f}_{s,ib}$	average sleeve friction over caisson tip and below bottom stiffener base
626	$\bar{f}_{s,it}$	average sleeve friction over bottom stiffener base penetration depth
627	G_s	specific gravity
628	G_{50}	average shear modulus
629	g	Earth's gravitational acceleration
630	H_c	open cavity depth

631	h	ring stiffener height
632	I_r	soil rigidity index
633	k	undrained shear strength gradient with depth
634	L	caisson length
635	LL	liquid limit
636	$N_{c,b}$	bearing capacity factor of bottom stiffener
637	$N_{c,f}$	bearing capacity factor of caisson full base
638	$N_{c,t}$	bearing capacity factor of caisson tip
639	N_{kt}	bearing capacity factor of piezocone
640	$N_{T\text{-bar}}$	bearing capacity factor of T-bar
641	n and n_1	factor relating operative shear strain rate to normalised velocity
642	PL	plastic limit
643	q_{cnet}	piezocone net tip resistance
644	$q_{cnet,b}$	piezocone net tip resistance at bottom stiffener base level
645	$q_{cnet,t}$	piezocone net tip resistance at caisson tip level
646	$\bar{q}_{cnet,t}$	piezocone (average) net tip resistance
647	R_c	piezocone radius
648	R_f	factor related to effect of strain rate and softening
649	R_{fe}	factor related to effect of strain rate and softening for end bearing resistance
650	R_{ff}	factor related to effect of strain rate and softening for frictional resistance
651	S_t	soil sensitivity

652	s	ring stiffener spacing (centre to centre)
653	s_u	undrained shear strength
654	s_{um}	undrained shear strength at mudline
655	$s_{u,e}$	undrained shear strength from penetrometer extraction test
656	$s_{u,p}$	undrained shear strength from penetrometer penetration test
657	$s_{u,p, T\text{-bar}}$	undrained shear strength from T-bar penetration test
658	$s_{u,0b}$	local undrained shear strength at bottom of stiffener base level
659	$s_{u,0t}$	local undrained shear strength at caisson tip level
660	s_{uH_c}	undrained shear strength at cavity base level i.e. at H_c
661	$\bar{s}_{u,o}$	average undrained shear strength over caisson tip penetration depth
662	$\bar{s}_{u,ib}$	average undrained shear strength over caisson tip and below bottom stiffener
663		base
664	$\bar{s}_{u,it}$	average undrained shear strength over bottom stiffener base penetration depth
665	T and T^*	dimensionless time
666	t	caisson wall thickness
667	t_c	reconsolidation time
668	t_d	dissipation time
669	U	normalised excess pore pressure
670	V	dimensionless velocity
671	v	object penetrating velocity
672	v_c	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	α_c	reduction factor for sleeve friction
681	α_0	friction factor for caisson outer wall-soil interface
682	α_{ib}	friction factor for caisson inner wall-soil interface below bottom stiffener
683	α_{it}	friction factor for caisson inner soil-soil interface along faces of stiffeners
684	β	rate parameter for power expression
685	Δu	excess pore pressure
686	Δu_{max}	maximum excess pore pressure
687	δ_{rem}	remoulded strength ratio
688	$\dot{\gamma}$	shear strain rate
689	$\dot{\gamma}_{ref}$	reference shear strain rate
690	γ'	effective unit weight of soil
691	θ_a	padeye load inclination
692	θ_0	mudline load inclination

693	ξ	cumulative plastic shear strain
694	ξ_{95}	cumulative plastic shear strain required for 95% remoulding

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Table 1. Model and prototype caisson dimensions

Dimension	Symbol	Model caisson	
		Model	Prototype (at 200g)
Diameter	D	40.00 mm	8.00 m
Length	L	120.00 mm	24.00 m
Thickness	t	0.40 mm	0.08 m
Aspect ratio	L/D	3.00	
Thickness ratio	D/t	100.00	
Ring stiffener width	b	2.50 mm	0.50 m
Ring stiffener height	h	1.00 mm	0.20 m
Ring stiffener spacing (c/c)	s	20.00 mm	4.00 m
Distance of bottom stiffener base from caisson tip	w	19.50 mm	3.90 m
Dry weight	W_d	1.50 N	12.01 MN
Submerged weight in water	W_s	1.00 N	8.05 MN
Submerged weight in soil	W_{ss}	0.68 N	5.40 MN

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Table 2. Summary of centrifuge tests conducted

Test	Installation			Time allowed for consolidation, t_c		Extraction					
	Method	Installation depth: m	Maximum installation resistance, $F_{p,max}$: MN	Model: min	Prototype: month	Extraction mode	Mudline load inclination, θ_0 : °	Padeye load inclination, θ_a : °	Holding capacity		
									$F_{e,max}$: MN	F_N : MN	F/W_d
T1	Jacking + suction	22.40	15.50	13.14	12	Sealed	90	90	22.60	22.60	2.77
T2		22.50	15.75	6.57	6	Sealed			21.10	21.10	2.64
T3		22.45	15.60	2.16	2	Sealed			19.28	19.28	2.49
T4		22.40	15.55	2.16	2	Vented			8.88	8.88	1.62
T5	Jacking	23.20	16.17	13.14	12	Sealed			22.84	22.84	2.79
T6		22.95	16.10	6.57	6	Sealed			21.40	21.40	2.67
T7		22.90	16.05	3.29	3	Sealed			19.94	19.94	2.54
T8		22.85	16.00	0.00	0	Sealed			17.08	17.08	2.31
T9		23.00	16.15	2.16	2	Vented			9.89	9.89	1.71
T10		22.85	16.01	13.14	12	Sealed	80	80	30.90	20.30	2.58
T11		22.90	16.07				40	40	41.59	30.98	3.47
T12		23.00	16.12				20	32	47.40	36.80	3.95
T13		23.00	16.15				0	15	57.55	46.95	4.80

842 Installation rate, v_p = extraction rate, v_e = 0.4 mm/s

843 **Number of Figure: 11**

- 844 **Figure 1.** Suction caisson anchor: (a) Schematic representation of installed suction
845 caisson anchor and idealised seabed strength profile (PPT: pore pressure
846 transducer; TPT: total pressure transducer); (b) Model stiffened caisson
- 847 **Figure 2.** Results from soil characterisation tests using T-bar and piezocone: (a)
848 Undrained shear strength profiles; (b) Undrained shear strength degradation
849 during T-bar cyclic sequence; (c) Pore water pressure (u_2 position) during
850 piezocone test; (d) Interpretation of c_h from piezocone dissipation data
- 851 **Figure 3.** Schematic representation of experimental arrangement for caisson installation
852 and pullout
- 853 **Figure 4.** Installation resistance profiles and theoretical upper and lower bound
854 predictions
- 855 **Figure 5.** Simplified installation mechanism following Zhou et al. (2016) (see also
856 Equations 2 and 3)
- 857 **Figure 6.** Theoretical predictions for existing data: (a) Field data of stiffened caisson
858 installation in calcareous silty clay (Erbrich & Hefer, 2002) and theoretical
859 upper and lower bound predictions; (b) Centrifuge test data of a stiffened
860 caisson installation in Gulf of Guinea clay (Gaudin et al., 2014) and theoretical
861 prediction
- 862 **Figure 7.** Profiles of net tip resistance, q_{cnet} , and sleeve friction, f_s , from piezocone test
- 863 **Figure 8.** Vertical extraction resistance profile ($\theta_a = \theta_0 = 90^\circ$, pullout from caisson top;
864 Tests T1~T9, Table 2)

865 **Figure 9.** Effect of reconsolidation time on friction factor, α_o , back-figured from
866 measured holding capacity

867 **Figure 10.** Caisson capacity under inclined loading: (a) Effect of mudline load inclination
868 θ_0 on extraction resistance (Tests T10~T13; Table 2); (b) Net holding capacity
869 under vertical and horizontal loading; (c) Failure envelope for vertical and
870 horizontal loading in normalised load space

871 **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)

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(b) Model stiffened caisson

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Figure 1. Suction caisson anchor

(a) Undrained shear strength profiles

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(b) Undrained shear strength degradation during T-bar cyclic sequence

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(c) Pore water pressure (u_2 position) during piezocone test

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(d) Interpretation of c_h from piezocone dissipation data

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Figure 2. Results from soil characterisation tests using T-bar and piezocone

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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)

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

6(b) Centrifuge test data of a stiffened caisson installation in Gulf of Guinea clay (Gaudin et al., 2014) and theoretical prediction

Figure 6. Theoretical predictions for existing data

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937 (a) Effect of mudline load inclination θ_0 on extraction resistance (Tests T10~T13; Table 2)

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(b) Net holding capacity under vertical and horizontal loading

(c) Failure envelope for vertical and horizontal loading in normalised load space

Figure 10. Caisson capacity under inclined loading

Figure 11. Dependence of reconsolidation time after installation on caisson capacity