T-bar to Spudcan Design Approach in Single Layer Clays and Calcareous Silts

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Abstract: Difficulties in obtaining high-quality soil samples from offshore sites and heavy involvement of lab testing and engineering judgement have necessitated increasing reliance on in situ penetrometer tests, giving continuous penetration resistance profiles. This paper reports a framework for estimating spudcan penetration resistance directly from continuous resistance profile of a full-flow penetrometer (T-bar) test in single layer clays and calcareous silts. The framework was validated chiefly against centrifuge test data, supplemented by some reported field data.

CE Database subject headings: Calcareous silts; Centrifuge models; Clays; Foundations; Load bearing capacity; Offshore structures; Penetration resistance; Strain rate; Strain softening.

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16 **Introduction**

17 Much of offshore design in fine grained sediments hinges on a design shear strength profile,
18 including identification of layers, which is generally based on results from laboratory or vane
19 shear strength data, increasingly supplemented by profiles of in-situ penetrometers test. With
20 the samples taken, various offshore or onshore laboratory tests are conducted from which
21 shear strength data and other engineering parameters are derived. Variability in the shear
22 strength measured from several types of tests, with potentially some degree of sampling
23 disturbance, leads to fundamental uncertainty that may in turn lead to inaccuracies in the
24 bearing capacity calculation process. Conversely, an in situ penetrometer test provides a
25 continuous penetration profile, which reduces these variability and subjectivity, but
26 interpretation of strength relies on selection of penetrometer factor. There is therefore
27 increasing interest in prediction of offshore foundations performance directly from in situ
28 penetrometer test results, just has been applied successfully in pile design. This paper reports
29 a framework for estimation of spudcan penetration profile directly from T-bar test in single
30 layer clays and silts, based on centrifuge test data, supplemented by the results from large
31 deformation finite element (LDFE) analyses. Factors affecting this scaling of the penetration
32 resistance for direct application in spudcan design are explored.

33 ***Jack-up Rig, Spudcan Foundations and T-bar Penetrometer***

34 Most of the world's offshore drilling in water depths up to 150 m is performed from mobile
35 jack-up rigs. The units consist of a buoyant triangular platform supported by three
36 independent vertically retractable *K*-lattice legs, each resting on a spudcan. Typically,
37 spudcans are 10 to 20 m (equivalent area) diameter and penetrate at a rate of 0.4 to 4 m/h.
38 Evolution of jack-up installation continues in single layer clay deposits (e.g. normally
39 consolidated (NC), lightly overconsolidated (LOC) and heavily overconsolidated (HOC)),
40 such as in the Gulf of Mexico, the North Sea and the Sunda Shelf (see Figure 1a) (Menzies

and Roper 2008; InSafeJIP 2010; Menzies and Lopez 2011). Progression is also occurring in calcareous sediments – prevalent in Australian waters (see Figure 1b) and in the Caspian Sea, Arabian Gulf, offshore Qatar, offshore Florida and South China Sea (Erbrich 2005; Menzies and Lopez 2011).

Novel full-flow penetrometers, T-bar and ball, tests are now being used increasingly as one of the site investigation tools in soil characterisation. A T-bar typically comprises a cylinder, 40 mm diameter and 250 mm long, attached at right angles to a shaft of 25 mm diameter. Penetration tests are carried out at rate of 20 mm/s. Due to mobilisation of ‘full-flow’ at shallow penetrations and very low area ratio (the cross-sectional area of the connection shaft to the projected area of the penetrometer in a plane normal to the shaft), to obtain net resistance, the measured penetration resistance requires minimal, or negligible, correction for the unequal area effect and the contribution to the T-bar resistance from the overburden stress (Randolph et al. 2007). Note, flow around mechanism is also mobilised during a spudcan penetration in clay (Hossain et al. 2006).

Conventional Design Approach for Spudcan Foundations

Penetration resistance of spudcan foundations is commonly assessed in the framework of bearing capacity formulations (InSafeJIP 2010; ISO 2012). This is a two-step approach in which soil strength parameters are derived from the site specific soil investigation data (comprising tests on borehole samples and in-situ penetrometer testing) for use in bearing capacity models. Penetrometer data provide a means of interpolating shear strengths, in terms of relative values, but the resistance factor relating penetration resistance to shear strength measured in an element test is generally derived for the given site by correlation. Thus the absolute value of the penetration resistance is not used directly (Randolph et al. 2007). Recently, a joint-industry project highlighted the necessity of scaling of penetrometer profiles for direct application in estimation of spudcan penetration resistance (InSafeJIP 2010). A field

penetrometer integrated with the jack-up unit is repeatedly emphasised, which will provide on-board penetration profiles prior to the jack-up installation (Quah et al. 2008).

T-Bar to Spudcan Direct Design Approach Framework

Hossain et al. (2006, 2014) and Hossain and Randolph (2009a) investigated spudcan penetration in clay through an extensive series of centrifuge model tests and LDFE analyses. In single layer clay, the observed soil flow patterns revealed three distinct mechanisms of soil flow around the advancing spudcan: (1) shallow failure with outward and upward flow leading to surface heave and formation of a cavity above the spudcan (Figure 2a); (2) gradual backflow into the cavity (Figure 2b); and (3) deep failure with fully localised flow around the embedded spudcan and unchanged cavity (Figure 2c). The continual backflow gradually provides a seal above the spudcan and limits the cavity depth (H_{cavS}). When the spudcan reaches a deep penetration depth (H_{dS}), a fully localised failure mechanism is attained. Hossain and Randolph (2009a) and Hossain et al. (2014) proposed a new design chart along with robust expressions to estimate these two critical depths for the penetrating spudcan as

$$\frac{H_{cavS}}{D_S} = S^{0.55} - \frac{S}{4} \quad \text{where} \quad S = \left(\frac{s_{um}}{\gamma' D_S} \right)^{1 - \frac{k}{\gamma'}} \quad (1)$$

$$\frac{H_{dS}}{D_S} = \frac{H_{cavS}}{D_S} + 0.7 \quad (2)$$

where s_{um} is the intact (undisturbed) soil strength at the seabed (mudline) and k is the rate of increase in strength (s_u) with depth. Penetration depth $d \leq H_{cavS}$ corresponds to mobilization of shallow failure mechanisms associated with shallow bearing capacity factor, N_{sS} ; while $d > H_{cavS}$ relates to mobilization of deep failure mechanisms associated with deep bearing capacity factor N_{dS} (where suffixes S, d and s indicate spudcan, deep and shallow

respectively). The limiting deep bearing capacity factor is attained at penetration depth $d > H_{ds}$.

For estimating the deep penetration depth for the T-bar penetrating clay (H_{dT}), White et al. (2010) suggested an expression as

$$\frac{H_{dT}}{D_T} = 2.58 \left(\frac{s_u H_{dT}}{\gamma' D_T} \right)^{0.46} + 0.24 \left(\frac{s_u H_{dT}}{\gamma' D_T} \right)^{-0.63} - 0.5 \quad (3)$$

The terms are defined under the notation section. For Equations 1, 3 and others, careful consideration should be given to an appropriate choice of shear strength. Based on the soil failure mechanism, the shear strength should be that representative of the average shear strength measured in triaxial compression, simple shear, and triaxial extension.

At shallow penetration, less than the limiting cavity depth ($d \leq H_{cavS}$), the vertical bearing capacity, Q_s , for spudcans can be calculated as (Hossain and Randolph 2009a; ISO 2012)

$$Q_s = s_u N_{Ss} A_s + \gamma' (A_s d + V_{sb}) \quad \text{for } d \leq H_{cavS} \quad (4)$$

For $d > H_{cavS}$, the vertical bearing capacity, Q_s , can be calculated according to (Hossain and Randolph 2009a; Hossain et al. 2014)

$$Q_s = s_u N_{Sd} A_s + \gamma' V_s \quad (5)$$

V_{sb} and V_s are the buried spudcan volumes in soil, as depicted in Figure 4 of Hossain et al. (2014). Shallow bearing capacity factor N_{Ss} for on-homogeneous fine grained soil expressed as

$$N_{Ss} = N_{Ss0} \left[1 + 0.161 \left(\frac{k D_s}{s_{u0}} \right)^{0.8} / \left(1 + \frac{d}{D_s} \right)^2 \right] \quad (6)$$

where N_{Ss0} is the bearing capacity factor for uniform clay, given by

$$N_{Ss0} = 5.69 + \frac{d}{0.2D_S} \left(1 - \frac{d}{2.85D_S} \right) \leq 9.2 \quad (7)$$

And deep bearing capacity factor N_{Sd} as

$$N_{Sd} = 10 \left(1 + 0.065 \frac{d}{D_S} \right) \leq N_{Sdl} = 11.3 \quad (8)$$

where N_{Sdl} is the limiting deep bearing capacity factor. White et al. (2010) proposed a similar expression for T-bar penetration in uniform clay

$$N_{Ts} = 2 + (N_{Td} - 2) + \left(\frac{H_{dT}}{d_{tipT}} \right)^p \leq N_{Td} = N_{Tdl} = 9.5 \quad \text{where } p = 0.61 \left(\frac{s_u}{\gamma' D_T} \right)^{-0.31} \quad (9)$$

where suffix T indicates T-bar. Note, as the T-bar penetration resistance is directly scaled, the undrained shear strength is unknown. Furthermore for soft fine grained soils, penetration resistance at shallow depth is not critical. Nevertheless, for calculating H_{cavS} , H_{dT} and shallow bearing capacity factors, N_{Ss} and N_{Ts} , a profile of undrained shear strength can be deduced by using $N_{Td} = N_{Tdl} = 11.87$ (as discussed later).

For predicting spudcan penetration profile directly from the T-bar penetrometer data in single layer clay and calcareous silt, a design framework is proposed rearranging Equations 4 and 5 adding some adjustment factors as

$$Q_S = \frac{q_T}{N_T} N_{Ss} A_S + \gamma' (A_S d + V_{Sb}) \quad (10a)$$

$$Q_S = (q_T - \gamma' z) \frac{F_{DS}}{F_{DT}} \frac{N_{Ss,ideal} F_{RSS}}{N_{Ts,ideal} F_{RST}} A_S + \gamma' (A_S d + V_{Sb}) \quad \text{for } d/D_S \leq H_{dT}/D_S \quad (10b)$$

$$Q_S = q_T \frac{F_{DS}}{F_{DT}} \frac{N_{Ss,ideal} F_{RSS}}{N_{Td,ideal} F_{RST}} A_S + \gamma' (A_S d + V_{Sb}) \quad \text{for } H_{dT}/D_S \leq d/D_S \leq H_{cavS}/D_S \quad (10c)$$

$$Q_S = q_T \frac{F_{DS}}{F_{DT}} \frac{N_{Sd,ideal} F_{RSS}}{N_{Td,ideal} F_{RST}} A_S + \gamma' V_S \quad \text{for } d/D_S > H_{cavS}/D_S \quad (11)$$

where q_T is the bearing pressure from T-bar penetration test, F_D is the drainage correction factor, $N_{S,ideal}$ and $N_{T,ideal}$ are respectively the bearing capacity factors of spudcan and T-bar for ideal non-softening, rate independent soil, and F_{RS} is the relative viscous rate and softening effect adjustment factor.

The proposed concept is consistent with the ones proposed by Erbrich (2005) and Randolph et al. (2007). All the correction factors in Equations 10 and 11 are to adjust T-bar behaviours to spudcan behaviours. These factors have been outlined in details below mainly in the context of deep penetration (Equation 11) as in soft fine grained soils spudcans penetrate 2~5 diameters during the preloading stage (Menzies and Roper 2008; Menzies and Lopez 2011).

135 ***Effect of Partial Drainage: F_D***

From the theoretical framework of Finnie and Randolph (1994), House et al. (2001), Randolph and Hope (2004) and Chung et al. (2006), the non-dimensional critical parameter that determines the degree of drainage and the effect of consolidation under a penetrating object is

$$V' = \frac{v D_e}{c_v} \quad (12)$$

where v is the penetration velocity, c_v the coefficient of consolidation of the soil and D_e the (projected area) equivalent diameter of the object. Using of an equivalent diameter for assessing drainage conditions (or consolidation rate) is suggested by Chung et al. (2006).

Figure 3 plots the backbone curves for T-bar and spudcans in clay and calcareous silt. For T-bar in kaolin clay, backbone curves with an expression were proposed by Watson and Suemasa (unpublished data, 2000) without accounting for viscous rate effect and by Randolph

and Hope (2004) with and without viscous rate effect in the partially drained region. Similar expressions can be derived based on the results presented by Cassidy (2012) and Finnie and Randolph (1994) for T-bar and spudcan (precisely flat footing and cone for Finnie and Randolph) in calcareous silt and by Barbosa-Cruz (2007) for spudcan in kaolin clay as (excluding rate effect)

$$\frac{q}{q_{\text{undrained}}} = a + \frac{b}{1 + cV'^m} \quad (13)$$

where a is the normalised bearing pressure in the undrained region ($q_{\text{undrained}}$), and as normalisation on the left side is carried out by $q_{\text{undrained}}$, $a = 1$. As V' approaches zero, a drained response is attained with a value of $a + b$. Parameters c and m indicate the stiffness of the curve or in another word breadth of the transition from undrained to drained zone. The values of parameters b , c , m for T-bar and spudcan penetration in kaolin clay and calcareous silt are given in Table 1.

It is suggested to use Cassidy (2012)'s curves for spudcan and T-bar penetration in calcareous silt and Randolph and Hope (2004)'s curve (excluding viscous rate effect) for T-bar and Barbosa-Cruz (2007)'s curve for spudcan penetration in clay. The corresponding correction factors F_{DS} and F_{DT} to be used in Equation 13 can be calculated depending on V' for T-bar and spudcan respectively. If for both objects $V' \geq 30$, i.e. they lie in the undrained region, $F_{\text{DS}} = F_{\text{DT}} = 1$. For calcareous silt, if $V' = 0.8$ for T-bar and $V' > 30$ for spudcan, the corresponding $F_{\text{DT}} = 4$ and $F_{\text{DS}} = 1$. It should be noted however, the results presented by Cassidy (2012) are for a particular type calcareous silt and for centrifuge T-bar to spudcan effective diameter ratio of 5.3. Further investigations are required for variable silt properties and for offshore T-bar to spudcan effective diameter ratio of 88~177 to make the corresponding curve and expression robust.

Strain Rate and Softening Effect: F_{RS}

In the undrained regime, the influence of the rate of loading on the undrained shear strength (and hence the penetration resistance) dominate the behaviour of a penetrating object. Most soils exhibit some viscous rate effect, which leads to higher undrained strengths for higher applied strain rates. This effect is commonly evaluated using a semi-logarithmic function (or power law relationship) as

$$\frac{s_u}{s_{u,ref}} = 1 + \mu \log \left(\frac{\dot{\xi}}{\dot{\xi}_{ref}} \right) \quad \text{in terms of shear strength} \quad (14)$$

$$\frac{q}{q_{ref}} = 1 + \mu \log \left(\frac{\frac{v}{D}}{\frac{v_{ref}}{D}} \right) \quad \text{in terms of penetration resistance} \quad (15)$$

where s_u , $s_{u,ref}$ are the undrained shear strength at strain rates of $\dot{\xi}$, $\dot{\xi}_{ref}$ and q , q_{ref} are the penetration resistance at penetration rates of v , v_{ref} respectively. The parameter μ is the rate of increase in shear strength (or penetration resistance) per log cycle increase in strain rate. Note that the physical diameter D , rather than D_e , is used in the viscous part since v/D determines the average strain rate ($\dot{\xi}$). Based on the results from field and centrifuge tests on normal clays, Chung et al. (2006), Low et al. (2008) and Hossain et al. (2014) reported $\mu = 0.15 \sim 0.21$ for T-bar and a lower value of $0.1 \sim 0.12$ for ball and ~ 0.1 for spudcan. The values for ball and spudcan are similar due to geometric resemblance – both are circular, axisymmetric and flow-around objects. Lower rate dependency of ball penetrometer is also evident from Figure 4, which shows a higher value of 0.32 (for T-bar) and 0.16 (for ball) for kaolin clay (Lehane et al. 2009). For T-bar penetration in the North-West Shelf calcareous clay, $\mu = 0.1$ and for silt, a much lower value of 0.03 was measured by Erbrich (2005).

Fine-grained soils also exhibit strain softening with gradual loss of strength when subjected to large shear strains, which suppresses the effect of strain rate. The combined effect of strain rate and strain softening on the response of T-bar and spudcan was investigated through large deformation FE analysis by Zhou and Randolph (2009) and Hossain and Randolph (2009b). They proposed an approximate expression for limiting deep bearing capacity factor of T-bar, ball and spudcan respectively as

$$N_{Td} = (1 + 4.8\mu) \left\{ \delta_{rem} + (1 - \delta_{rem}) e^{-1.5(3.7/\xi_{95})} \right\} N_{Td,ideal} \quad (16)$$

$$N_{Bd} = (1 + 4.8\mu) \left\{ \delta_{rem} + (1 - \delta_{rem}) e^{-1.5(3.3/\xi_{95})} \right\} N_{Bd,ideal} \quad (17)$$

$$N_{Sd} = (1 + 1.47\mu) \left\{ \delta_{rem} + (1 - \delta_{rem}) e^{-1.5(4.8/\xi_{95})} \right\} N_{Sd,ideal} \quad (18)$$

where $N_{Sd,ideal} = 11.3$ (regardless of roughness) and δ_{rem} denotes the ratio of fully remoulded to initial shear strength (i.e. the inverse of the sensitivity, S_t). For slightly rough (with interface roughness $\alpha = 0.2$) T-bar and ball without any shaft (i.e. ratio of shaft area to penetrometer projected area, $a = 0$), Zhou and Randolph (2009) reported $N_{Td,ideal} = 9.95$ and $N_{Bd,ideal} = 12.01$. Recently, Zhou and Randolph (2011) and Zhou et al. (2013) investigated the effect of shaft on resistance of a penetrometer and proposed an adjustment factor as a function of area ratio a . An according adjustment gives $N_{Td,ideal} = 9.5$ (centrifuge T-bar with $a = 0.16$), 9.8 (field T-bar with $a = 0.05$) and $N_{Bd,ideal} = 11.5$ (centrifuge ball with $a = 0.25$), 11.2 (field ball with $a = 0.11$). Although this paper focuses on T-bar and spudcan, a few centrifuge tests were carried out on ball (as discussed later) and hence equation/parameters for ball are also included. ξ_{95} represents the cumulative total absolute shear strain required for 95% remoulding, thus reflecting the relative ductility of the soil. The results for $\mu = 0 \sim 0.2$, $\xi_{95} = 10 \sim 15$ and no-softening, $\delta_{rem} = 0.2 \sim 0.25$, are shown in Figure 5.

The rate gradient $R_T = 4.8$ for the T-bar implies that the average rate of strain experienced by soil flowing past penetrometer is some $10^{4.8}$ (i.e. five orders of magnitude) greater than the reference value. This is significantly lower, and only $10^{1.47}$ ($R_s = 1.47$) i.e. 1.5 orders of magnitude greater for spudcan. For softening, the value of $\xi_T = 3.7$ represents an average cumulative shear strain (of 370%) experienced by soil particles as they traverse the flow mechanism around T-bar. This is slightly higher for spudcan of $\xi_s = 4.8$ (i.e. 480%). In essence, T-bar is highly sensitive to both strain rate and strain softening while spudcan is less sensitive to the former but has similar sensitivity to the latter, as also mapped by Randolph et al. (2007) (see Figure 6).

For calculating correction factor F_{RS} , it is suggested using Equations 16~18 neglecting the 2nd bracketed terms (as the corresponding influence is insignificant). For clay, calcareous silt and calcareous clay, rate parameter μ_s or $\mu_B = 0.12, 0, 0.05$ (for spudcan or ball) and $\mu_T = 0.18, 0.03, 0.1$ (for T-bar), respectively, may be used. The rate dependent term 4.8, 4.8, 1.47 in Equations 16~18 were obtained considering reference $(v_T/D_T)_{ref} = 0.5$, $(v_B/D_B)_{ref} = 0.25$, $(v_s/D_s)_{ref} = 3.09 \times 10^{-5}/s$ respectively. An adjustment of these coefficients is required according to the corresponding penetrating rate in the centrifuge or field. An equivalent resistance factor for other normalised penetration rates velocities may be obtained by adjusting the rate dependent term accordingly by the logarithm of the ratio of normalised penetration rates. For instance, for spudcan penetration with $v_s/D_s = 0.0067/s$, such adjustment leads to the rate dependent term of $(1.47 + \log(0.0067/3.09 \times 10^{-5})) = 3.34$.

Although Equations 16~18 were developed for limiting deep bearing capacity factors, they may also be employed for shallow bearing capacity factors. The proposed framework is validated below against centrifuge test results and reported field data.

Centrifuge Tests

Model spudcans and T-bar penetration tests were carried out at a gravitational acceleration of 50 to 200g in the drum centrifuge at the University of Western Australia (Stewart et al. 1998). Test specimens were confined to a 258 (length) $\times$ 163 (width) $\times$ 160 (depth) mm strongbox that was mounted within the annular channel of the drum centrifuge (Hossain and Randolph 2010). On each box, one spudcan and two T-bar tests, immediately before and after the spudcan penetration test, were performed. A 20~30 mm deep (in model scale) water layer was maintained above the soil surface during each test.

Spudcan and T-bar models

Model tests were performed to measure the load-penetration response using two full-spudcans of 30 and 60 mm diameter ($D_s = D_{se} = 30$ and 60 mm; where suffix e indicates equivalent area; Figure 7). To reveal the soil failure mechanisms around the advancing spudcan, tests were also carried out on a half-spudcan model of 60 mm diameter. The models were made from duraluminium and included a 13° shallow conical underside profile (included angle of 154°) and a 76° protruding spigot. The shape was chosen similar to the spudcans of the ‘Marathon LeTourneau Design, Class 82-SDC’ jack-up rig, as illustrated by Menzies and Roper (2008).

The T-bar penetrometer (Stewart and Randolph 1991) is shown on the right side of Figure 7. It comprises a lightly sand-blasted cylinder, 5 mm diameter and 20 mm long ($D_T = 5$ mm and $D_{Te} = 11.3$ mm), attached to a shaft that tapers down to 4.5 mm diameter.

Selection of Parameters

The experimental program concentrated on simulating strength profiles from reported case histories, with similar normalised layer soil strength to the offshore cases histories reported by

Erbrich and Hefer (2002), Menzies and Roper (2008) and Menzies and Lopez (2011). Tests were carried out on reconstituted samples prepared using commercially available kaolin clay (KC). The results on calcareous clay (CC) and calcareous silt (CS15, CS20) obtained from the North-West Shelf, offshore Australia are also assembled from Watson (1999) and Cassidy (2012). The properties of the soils are given in Table 2.

Table 3 provides a summary of conducted full-spudcan tests in kaolin clay. The full-spudcan (or ball) tests carried out in calcareous clay and silt are tabulated in Table 4 (Watson 1999; Cassidy 2012). A total of 14 tests were reported linking T-bar to spudcan (or ball) penetration response on single layer (a) NC kaolin clay, (b) LOC kaolin clay, (c) HOC kaolin clay, (d) NC calcareous clay, (e) LOC calcareous clay, and (f) NC calcareous silts.

Preparation of Specimen

Kaolin clay was used for Events KC1~KC3. A homogeneous slurry was prepared by mixing commercially available kaolin powder. For Events KC1 and KC2 (Table 3), normally consolidated clay specimens with an increasing strength profile were prepared by performing consolidation directly in the drum centrifuge. To produce LOC specimens for Event KC2, the top few mm was carefully trimmed off the specimens (Hossain et al. 2006). HOC sample for Event KC3 was prepared by consolidating kaolin slurry at 1 g in a cell. Test specimen was prepared by cutting the pre-consolidated sample according to the size of the strongbox and planned layer thickness, and then placing the layer in the strongbox (Hossain and Randolph 2010).

Calcareous clay samples (Events CC1 and CC2; Table 4) were prepared in a similar manner, as described by Watson (1999). For Events CS(20)1 and CS(15)2, the bulk seabed samples were dried and sieved. To replace the fines believed to have been lost during the drying process, 15~20% of the soil constituted commercially available OMYACARB (as such

referred to as ‘CS(15)’ or ‘CS(20)’). The silt, OMYACARB and silicon oil were mixed whilst being de-aired under a vacuum of 100 kPa. The slurry was then consolidated and tested in the UWA beam centrifuge (Watson 1999; Cassidy 2012).

Penetration Rate

All tests were carried out using displacement controlled system. For spudcan tests, a load cell fitted in between the spudcan shaft and the actuator loading arm measured load-penetration responses and for T-bar tests, the load is measured just behind the cylindrical bar. The penetration rate v , normalised velocity index $V' = vD_e/c_v$ (where v is the penetration velocity, D_e the object effective diameter and c_v the consolidation coefficient) and nominal strain rate v/D for various probes are given in Table 5. These ensured undrained behaviour in the investigated soils as will be discussed later.

Results

Full-spudcan (or ball) coupled with T-bar tests (Tables 3 and 4) were undertaken within the body of the test specimens in order to measure the penetration resistances. The results are presented in terms of ultimate bearing pressure, $q_u = P/A$ (where P is the penetration resistance and A is the largest plan area of the object). The results are discussed below for each group of profile, focusing on establishing direct relation between spudcan and T-bar penetration resistance profiles. The soil failure mechanisms from half-spudcan tests (carried out against the window of the strongbox) will be illustrated to provide observation wherever necessary.

NC Clay (Event KC1)

Three tests were carried out on NC clay ($D_{\text{Spe}} = D_{\text{Sp}} = 3$ m, $D_{\text{Tpe}} = 1.13$ m and $D_{\text{Tp}} = 0.5$ m; where suffix p indicates equivalent size in prototype; Event KC1; Table 3). The tests in this event simulate the seabed sediments in the Gulf of Mexico (e.g. see Figure 1a). Figures 8a and 8b show the load-penetration responses for Tests FS1~FS3 (Table 3) as a function of (normalised) tip penetration, d/D_{Spe} , and LRP penetration, d/D_{Spe} respectively. This is to show the effect of different distance of LRP from the tip of the spudcan (0.7 m = $0.23D_{\text{Spe}}$) or invert of the T-bar (0.25 m = $0.5D_{\text{Tp}}$). In Figure 8a, the profiles all start from (0, 0) origin in NC clay. In Figure 8b, they lifted up by different depths (see Figure 7b), and the spudcan penetration resistances show slightly higher bearing pressure at the mudline ($d/D_{\text{Spe}} = 0$; all subsequent penetration response figures have been plotted starting from this point). This disparity is mainly due to the geometric contrast between spudcan and T-bar - axisymmetric vs plane strain.

NC and LOC Calcareous Clay (Event CC1 and CC2)

The seabed sediments with calcareous soils are prevalent in offshore Australia, the Arabian Gulf and Caspian Sea and in some regions of the Gulf of Mexico and South China Sea (e.g. see Figure 1b). In absence of spudcan penetration data in calcareous clay, the load-penetration responses from ball and T-bar penetration tests are plotted in Figure 9 (Tests B1 and B2, Event CC1 and CC2; Table 4).

NC Calcareous Silt (Event CS(20)1 and CS(15)2)

In calcareous normally consolidated silt (Event CS(20)1 and CS(15)2; Table 4), tests were carried out at various penetration rates. A spudcan of $D_{\text{Spe}} = 6$ m was penetrated at rates of $v_{\text{S}} = 1$ and 2 mm/s, a ball of $D_{\text{Bpe}} = 1.8$ mm at $v_{\text{B}} = 1$ mm/s and T-bar of $D_{\text{Tpe}} = 1.13 \sim 1.7$ at $v_{\text{T}} = 1 \sim 3$ mm/s. However, all rates ensured undrained behaviour in the silts, with $V' = vD_{\text{pe}}/c_v =$

47.244~252.1 > 30 (see Table 5, discussed later). The results from T-bar, spudcan (after $d/D_{\text{Spe}} = \sim 0.3$) and ball tests are shown in Figure 10.

LOC Kaolin Clay (Event KC2)

Figure 11 shows the results from three tests conducted at different gravity levels (and hence prototype diameters are different) in soft lightly over consolidated kaolin clay (Tests FS4~FS6, Event KC2; Table 3). These LOC deposits represent the reported case histories from different locations in the Gulf of Mexico (e.g. see Figure 1a).

HOC Kaolin Clay (Event KC3)

Heavily overconsolidated clay deposits are commonly encountered in the Gulf of Mexico, North Sea and Sunda Shelf (e.g. see Figure 1a). In HOC kaolin clay with a relatively higher strength at the mudline (precisely higher normalised strength, $s_{\text{um}}/\gamma' D_{\text{Spe}}$; Event KC3; Table 3), the (absolute) depths of soil backflow and deep localised flow were higher for the spudcans than those for the T-bar (Table 3; Equations 2 and 3; also see Figure 12b). The cavity depth above the spudcan was relatively lower and negligible in soft LOC and NC clay, respectively (see Figures 2 and 12a). The influence of different mobilised soil failure patterns can be marked in the penetration resistance profiles illustrated in Figure 13. Bearing pressure from the spudcan penetration test is ~10% lower compared to that from the T-bar test. However, the gap reduces with increasing depth and finally the profiles merge together at deep penetrations of $d/D_{\text{Spe}} > 1.2$, which marks the attainment of a fully localised failure mechanism for the spudcan (see Table 3). Note, at the mudline, the spudcan bearing pressure is higher, which is consistent with the identified disparity in NC clay.

Discussion: Proposed Design Framework vs Centrifuge Test Data

All necessary parameters and calculated values of F_D and F_{RS} relative to the centrifuge tests are tabulated in Table 5.

NC and LOC Clays

For the tests on NC and LOC clays, $F_{DS} = F_{DT} = 1$, $F_{RSS} = 1.46$, $F_{RST} = 1.79$, $N_{Sdl,ideal} = 11.3$ and $N_{Td,ideal} = 9.5$ (Table 5). For NC clay (Tests FS1~FS3), $H_{cavS}/D_{Spe} = 0$, $H_{dT}/D_{Spe} = 0.05$ and Equation 11 gives $Q_s/A_s = q_u = q_t 0.97 + \gamma' V_s/A_s = q_t 0.97 + 2.18$ kPa (Table 3). The calculated profile is in excellent agreement with measured spudcan penetration data, as typical shown in Figure 8b. Similar calculations for Tests FS4, FS6 and FS7 also result in improved predictions at both shallow and deep penetrations (see Figures 11a and 11b).

HOC Clay

Calculation for OC clay (Test FS8) is slightly complicated. It consists of three stages. For $d/D_{Spe} \leq H_{dT}/D_{Spe}$, the ratio $N_{Ss,ideal}/N_{Ts,ideal}$ is > 1 and for $H_{dT}/D_{Spe} \leq d/D_{Spe} \leq H_{cavS}/D_{Spe}$ the ratio $N_{Ss,ideal}/N_{Td,ideal}$ is < 1 . Finally, the ratio $N_{Sd,ideal}/N_{Td,ideal}$ becomes > 1 again for $d/D_{Spe} > H_{cavS}/D_{Spe}$. Although the calculated profile shows a jump between 1st and 2nd stages in Figure 13 (and hence a smoothing may be necessary), it shows good agreement with the measured profile. The jump is associated with the switching from one expression (Equation 10b) to the other (Equation 10c).

Calcareous Clay

For the tests on calcareous clay (Tests B1 and B2), F_{DB} and F_{DT} can be assumed as 1 although no relevant data is available. $F_{RSB} = 1.22$, $F_{RST} = 1.44$, $N_{Bd,ideal} = 11.5$ and $N_{Td,ideal} = 9.5$ (Table 5). For NC clay (Test B1) with negligible depth required for attaining a deep failure mechanism, these give a ratio of $(F_{RSB}N_{Bd,ideal})/(F_{RST}N_{Td,ideal}) = 1.026$. The predicted profiles

as included in Figure 9 are consistent with the measured ones. Note, for LOC clay, H_{dT} and H_{dB} can be calculated using Equation 3 (of this paper) and Equation 4 (of Zhou et al. 2013).

It is apparent from Figures 8b and 9 that T-bar penetration resistance in the normally consolidated calcareous clay is more than double of that in the normally consolidated kaolin clay. This difference corresponds to mainly much stronger calcareous clay.

Calcareous Silt

For NC calcareous silt, the depth of attaining limiting capacity is negligible. The values tabulated in Table 5 give $(F_{RSS}N_{Sd,ideal})/(F_{RST}N_{Td,ideal}) = 1.03\sim 1.05$ and $(F_{RSB}N_{Bd,ideal})/(F_{RST}N_{Td,ideal}) = 1.05\sim 1.07$. The calculated profiles predicted the measured resistance very well (see Figure 10).

Exercise for Partial Drainage and Viscous Rate Effect

For the results from full-spudcan in kaolin clay presented so far in Figures 8, 11 and 13, the normalised velocity index $V' = vD_e/c_v$ ranges from 73.17 to 146.36, which lie around the trough of the backbone curve (see Figure 3 and also Chung et al. 2006; Low et al. 2008). It remains constant with a value of 137.81 for the T-bar penetration. As such, the drainage correction factor was taken as $F_{DS} = F_{DT} = 1$. In order to check the performance of the analogy for partially drained and high viscous rate effect zone, results from very high and low penetration velocity are assembled in this section. Barbosa-Cruz (2007) carried out tests at 200g using a 60 mm diameter spudcan ($D_s = 60$ mm) in the same kaolin clay varying penetration rate between 1 and 0.003 mm/s. A T-bar of dimensions identical to this study was used, and also was penetrated at an identical rate of 1 mm/s. The spudcan penetration results

for two extreme penetration rates ($v_s = 0.003$ and 1 mm/s) and for the rate identical to this study ($v_s = 0.2$ mm/s) are plotted in Figure 14, along with the T-bar resistance profiles.

Consistent to the results from this study, the spudcan penetration curve for $v_s = 0.2$ mm/s is similar to the results from T-bar tests particularly after adjusting for $(F_{RSS}N_{S,ideal})/(F_{RST}N_{T,ideal})$, as shown in Figure 14. However, the response for $v_s = 0.003$ mm/s is significantly higher, and the gap increases with penetration depth. This reflects the influence of two counterbalancing effects including (i) partial consolidation occurred for the slower rate of the spudcan and (ii) strain rate effect associated with the T-bar penetration (viscous region, $V' > 30$). For the spudcan penetration depth of $d/D_{spe} = 1$ and 1.44 , the normalised velocity index can be calculated as $V' = 1.75$ and 1.58 , respectively (as c_v varies with depth, see Table 2 and Randolph and Hope 2004; Barbosa-Cruz 2007). The corresponding correction, using the backbone curve for spudcan (see Figure 3 and Table 1; Barbosa-Cruz 2007), can be calculated as $F_{DS} = 2.65$ and 2.78 ($F_{DT} = 1$). The proposed approach gives the ratio $F_{DS}(F_{RSS}N_{S,ideal})/F_{DT}(F_{RST}N_{T,ideal}) = 2.12$ and 2.22 respectively. The calculated profile underestimates the measured data by $\sim 8\%$ at deep penetration depths ($d/D_{spe} > 1$). This may be due to the adopted c_v values, and hence V' , for deep penetration depths (considering that the backbone curve of Barbosa-Cruz 2007 is appropriate).

The result for $v_s = 1$ mm/s is also higher, but the difference is somewhat uniform over the full penetration depth. With $V' > 525$, this shows the effect of relative strain rate. The ratio for deep penetration can be calculated as $F_{DS}(F_{RSS}N_{S,ideal})/F_{DT}(F_{RST}N_{T,ideal}) = \sim 1$. The predicted profile using the proposed framework accounting for shallow bearing capacity factors and contribution from the spudcan volume leads to a reasonable estimate of measured profile, as shown in Figure 14. These exercises confirm the workability of the proposed analogy over a wider range of drainage conditions.

It should be noted though, in the field, depending on the soil type, similar correction may mostly be required for the penetrometer penetration due to its lower size, but the same principle is valid.

Comparison with Field Data

Menzies and Roper (2008) reported measured load-penetration records from various jack-up installations at thirteen locations in the Gulf of Mexico. The profiles for sites 1, 2 and 7 (in NC and LOC) and sites 4 and 5 (in HOC) are plotted in Figures 15a and 15b, respectively. At each of the sites, single boring was drilled and sampled. Soils were normally, lightly and heavily overconsolidated clays with strength mostly increased linearly with depth. Undrained shear strength profiles were presented based mainly on unconsolidated undrained triaxial tests. Coefficient of consolidation values ranging from 0.7 to 3.4 m²/h are typical for the soils of the sites. The T-bar penetration resistance (q_T) profiles were back calculated from the proposed undrained shear strength profiles dividing by N_{Td} . Collating a worldwide high quality database of lightly overconsolidated clays, Low et al. (2010) suggested an average T-bar factor of 11.87 based on triaxial compression, simple shear and triaxial extension strength for the estimation of intact undrained shear strength from the penetration resistance. As such $N_{Td} = 11.87$ was used in those back calculation.

In the field, the standard T-bar ($D_T = 40$ mm) penetration tests are carried at a rate of $v_T = 20$ mm/s. For the diameter of the spudcans for Sites 1 and 2 were 13.5 m, and that of the spudcans for Sites 4, 5 and 7 were 12 m. The penetration rate for the spudcans can be assumed as 2 m/h (InSafeJIP 2010). These values ensured undrained conditions for both the T-bar and spudcans, and provide $F_{DS}(F_{RSS}N_{S,ideal})/F_{DT}(F_{RST}N_{T,ideal}) = \sim 0.765$ (for Sites 1 and

2) and ~ 0.77 (for Sites 4, 5 and 7). Figure 15a shows a close agreement between the predicted and measured profiles.

For HOC clays, the critical depths H_{cavS} and H_{dT} can be calculated using Equations 1 and 3 as 0.8 and 5.8 m respectively (for Site 4: $s_u = 19.2 + 1.46z$ kPa) and 0.7 and 5.3 m respectively (for Site 5: $s_u = 15.6 + 1.24z$ kPa). The depth of 0.7~0.8 m is negligible, but that of 5.3~5.8 m in HOC clay is considerable. As such, shallow bearing capacity factors for spudcan were needed to be taken into account for $d \leq H_{cavS}$. Equation 10c was used for $d \leq H_{cavS}$ and Equation 11 for $d > H_{cavS}$. The predicted profiles well estimated the measured profiles.

Erbrich (2005) reported a case history at Yolla, offshore Australia. The spudcan foundations were 18.2 m in diameter. The site consisted of stratified soils with layers of calcareous silt and clay. The coefficients of consolidation for the clay and silt layers were respectively of 4000 and 9 m²/h. The standard T-bar penetration test was included in the site specific soil investigation program.

Again a spudcan penetration rate of 2 m/h was assumed. For the clay, both the T-bar and spudcan penetration fall in the undrained region whereas for silt, the T-bar penetration falls in the partially drained region. The profile from a T-bar test was scaled using Equation 11 and plotted in Figure 16 along with the recorded spudcan penetration data, with reasonable prediction evidenced.

Conclusions

This paper has reported a framework for estimating spudcan penetration resistance directly from in situ site investigation carried out by a full-flow penetrometer – T-bar. The framework was developed accounting for the influence of relative (a) drainage conditions, (b) induced strain rate and softening, (c) mobilising soil failure mechanisms and corresponding shallow

and deep bearing capacity factors. The scaled profile using the proposed framework estimated centrifuge test data and reported field data reasonably well.

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481   **Notation**

482	A	plan area at largest section
483	A_s	spudcan plan area at largest section
484	a	normalised bearing pressure in undrained region
485	b	difference of normalised bearing pressure between drained and undrained
486		region
487	c, m	indicator of stiffness of backbone curve or breadth of partially drained zone
488	c_v	coefficient of consolidation
489	D_B	ball diameter
490	D_{Be}	ball equivalent (area) diameter
491	D_{Bp}	ball diameter in prototype
492	D_{Bpe}	ball equivalent (area) diameter in prototype
493	D_e	equivalent (area) diameter
494	D_s	spudcan diameter at largest section
495	D_{Se}	spudcan equivalent (area) diameter
496	D_{Spe}	spudcan diameter in prototype
497	D_{Spe}	spudcan equivalent (area) diameter in prototype
498	D_T	T-bar diameter
499	D_{Te}	T-bar equivalent (area) diameter

500	D_{Tp}	T-bar diameter in prototype
501	D_{Tpe}	T-bar equivalent (area) diameter in prototype
502	d	penetration depth of load reference point (LRP) of spudcan and T-bar
503	d_{tip}	penetration depth of tip of spudcan or invert of T-bar
504	F_D	drainage correction factor
505	F_{DB}	drainage correction factor for ball
506	F_{DS}	drainage correction factor for spudcan
507	F_{DT}	drainage correction factor for T-bar
508	F_{RS}	viscous rate and softening effect adjustment factor
509	F_{RSB}	viscous rate and softening effect adjustment factor for ball
510	F_{RSS}	viscous rate and softening effect adjustment factor for spudcan
511	F_{RST}	viscous rate and softening effect adjustment factor for T-bar
512	G_s	specific gravity
513	H_{cavS}	open cavity depth after spudcan installation (taken as identical to back-flow
514		depth)
515	H_{dS}	depth to attainment of fully localised deep failure around spudcan
516	H_{dT}	depth to attainment of fully localised deep failure around T-bar
517	I_p	plasticity index
518	k	rate of increase of undrained shear strength

519	N_B	bearing capacity factor of T-bar
520	N_{Bd}	deep bearing capacity factor of ball
521	N_{Bdl}	limiting deep bearing capacity factor of ball
522	$N_{Bd,ideal}$	deep bearing capacity factor of ball for ideal non-softening, rate independent
523		soil
524	N_S	bearing capacity factor of spudcan
525	N_{Sd}	deep bearing capacity factor of spudcan
526	N_{Sdl}	limiting deep bearing capacity factor of spudcan
527	N_{Ss}	shallow bearing capacity factor of spudcan
528	N_{Ss0}	bearing capacity factor of spudcan for uniform clay
529	$N_{S,ideal}$	shallow bearing capacity factor of spudcan for ideal non-softening, rate
530		independent soil
531	$N_{Sd,ideal}$	deep bearing capacity factor of spudcan for ideal non-softening, rate
532		independent soil
533	$N_{Sdl,ideal}$	limiting deep bearing capacity factor of spudcan for ideal non-softening, rate
534		independent soil
535	N_T	bearing capacity factor of T-bar
536	N_{Td}	deep bearing capacity factor of T-bar
537	N_{Tdl}	limiting deep bearing capacity factor of T-bar
538	N_{Ts}	shallow bearing capacity factor of T-bar

539	$N_{T,ideal}$	bearing capacity factor of T-bar for ideal non-softening, rate independent soil
540	$N_{Td,ideal}$	deep bearing capacity factor of T-bar for ideal non-softening, rate independent
541		soil
542	P	penetration resistance
543	Q_s	force required to penetrate spudcan
544	Q_v	vertical penetration resistance
545	q	penetration resistance at any penetration rate v
546	q_{ref}	penetration resistance at reference penetration rate v_{ref}
547	q_T	T-bar penetration resistance
548	q_u	ultimate bearing pressure
549	$q_{undrained}$	undrained bearing pressure
550	R_B	rate gradient for ball
551	R_S	rate gradient for spudcan
552	R_T	rate gradient for T-bar
553	S_t	soil sensitivity
554	s_u	undrained shear strength of soil (at any strain rate $\dot{\xi}$)
555	$s_{uH_{dT}}$	undrained shear strength of soil at H_{dT}
556	s_{um}	undrained shear strength of soil at mudline
557	$s_{u,ref}$	undrained shear strength of soil at reference strain rate $\dot{\xi}_{ref}$

558	s_{u0}	local undrained shear strength of soil at spudcan base level
559	V_S	total volume of spudcan beneath backfill (buried volume)
560	V_{Sb}	volume of embedded spudcan below maximum bearing area
561	V'	non-dimensional velocity
562	v	object penetration rate
563	v_B	ball penetration rate
564	$(v_B/D_B)_{ref}$	representative reference strain rate of ball
565	v_{ref}	reference penetration rate
566	v_S	spudcan penetration rate
567	$(v_S/D_S)_{ref}$	representative reference strain rate of spudcan
568	v_T	T-bar penetration rate
569	$(v_T/D_T)_{ref}$	representative reference strain rate of T-bar
570	z	depth below mudline
571	δ_{rem}	ratio of fully remoulded to initial shear strength (the inverse of the sensitivity)
572	γ'	soil effective unit weight
573	γ'_{avg}	average soil effective unit weight
574	μ	rate parameter
575	μ_B	rate parameter for ball
576	μ_S	rate parameter for spudcan

577	μ_T	rate parameter for T-bar
578	σ'_{v0}	vertical effective stress
579	ξ	cumulative plastic shear strain
580	ξ_S	cumulative plastic shear strain of spudcan
581	ξ_T	cumulative plastic shear strain of T-bar
582	ξ_{95}	softening parameter (cumulative plastic shear strain required for 95%
583		remoulding)
584	$\dot{\xi}$	shear strain rate
585	$\dot{\xi}_{ref}$	reference shear strain rate
586		

References

- Barbosa-Cruz, E. R. (2007). “*Partial consolidation and breakthrough of shallow foundations in soft soil.*” Ph. D. Thesis, The University of Western Australia.
- Cassidy, M. J. (2012). “Experimental observations of the penetration of spudcan footings in silt.” *Géotechnique*, 62(8), 727-732.
- Chung, S. F., Randolph, M. F., and Schneider, J. A. (2006). “Effect of penetration rate on penetrometer resistance in clay.” *J. Geotechnical and Geoenvironmental Engineering*, ASCE, 132(9), 1188-1196.
- Erbrich, C. T. (2005). “Australian frontiers – spudcans on the edge.” *Proc Int. Symp. Frontiers in Offshore Geotechnics, ISFOG*, Perth, pp. 49-74.
- Erbrich, C. T., and Hefer, P. (2002). “Installation of the Laminaria suction piles – a case history.” *Proc. Offshore Technology Conference*, Houston, OTC 14240.
- Finnie, I. M. S., and Randolph, M. F. (1994). “Punch-through and liquefaction induced failure of shallow foundations on calcareous sediments.” *Proc. Int. Conf. on Behavior of Offshore Structures*, Boston, pp. 217-230.
- Hossain, M. S., Randolph, M. F., Hu, Y., and White, D. J. (2006). “Cavity stability and bearing capacity of spudcan foundations on clay.” *Proc. Offshore Technology Conference*, Houston, OTC 17770.
- Hossain, M. S., and Randolph, M. F. (2009a). “New mechanism-based design approach for spudcan foundations on single layer clay.” *J. Geotechnical and Geoenvironmental Engineering*, ASCE, 135(9), 1264-1274.

608 Hossain, M. S., and Randolph, M. F. (2009b). "Effect of strain rate and strain softening on the
609 penetration resistance of spudcan foundations on clay." *Int. J. Geomechanics, ASCE*
610 9(3), 122-132.

611 Hossain, M. S., and Randolph, M. F. (2010). "Deep-penetrating spudcan foundations on
612 layered clays: centrifuge tests." *Géotechnique*, 60(3), 157-170.

613 Hossain, M. S., Zheng, J., Menzies, D., Lillian, M., and Randolph, M. F. (2014). "Spudcan
614 penetration analysis for case histories in clay." *J. Geotechnical and Geoenvironmental*
615 *Engineering, ASCE*, 140(7), 04014034-1-13.

616 House, A. R, Oliveira, J. R. M. S., and Randolph, M. F. (2001). "Evaluating the coefficient of
617 consolidation using penetration tests." *Int. J. Physical Modelling in Geotechnics*, 1(3),
618 17-25.

619 InSafeJIP (2010). "*Improved guidelines for the prediction of geotechnical performance of*
620 *spudcan foundations during installation and removal of jack-up units.*" Joint Industry
621 Funded Project.

622 ISO (2012). "*Petroleum and natural gas industries – Site specific assessment of mobile*
623 *offshore units – Part 1: Jack-ups.*" International Organization for Standardization, ISO
624 19905-1.

625 Lehane, B. M., O'loughlin, C. D., Gaudin, C., and Randolph, M. F. (2009). "Rate effects on
626 penetrometer resistance in kaolin." *Géotechnique*, 59(1), 41-52.

627 Low, H. E., Lunne, T., Andersen, K. H., Sjørsen, M. A., and Randolph, M. F. (2010).
628 "Estimation of intact and remoulded undrained shear strengths from penetration tests in
629 soft clays." *Géotechnique*, 60(11), 843-859.

630 Low, H. E., Randolph, M. F., DeJong, J. T., and Yafrae, N. J. (2008). "Variable rate full-flow
631 penetration tests in intact and remoulded soil." *Proc. 3rd Int. Conf. on Geotechnical and*
632 *Geophysical Site Characterization*, Taylor & Francis Group, Taipei, Taiwan, pp. 1087-
633 1092.

634 Menzies, D., and Roper, R. (2008). "Comparison of jackup rig spudcan penetration methods
635 in clay." *Proc. Offshore Technology Conference*, Houston, OTC 19545.

636 Menzies, D., and Lopez, C. R. (2011). "Four atypical jack-up rig foundation case histories."
637 *Proc. 13th Int. Conf. The Jack-up Platform: Design, Construction and Operation*,
638 London.

639 Quah, C. K., Seng, F. K., Purwana, O. A., Keizer, L., Randolph, M. F., and Cassidy, M. J.
640 (2008). "An integrated in-situ soil testing device for jack-up rigs." *Proc. 2nd Jack-up*
641 *Asia Conference and Exhibition*, Singapore.

642 Randolph, M. F., and Hope, S. (2004). "Effect of cone velocity on cone resistance and excess
643 pore pressures." *Proc. Int. Symp. Engineering Practice and Performance of Soft*
644 *Deposits*, Osaka, pp. 147-152.

645 Randolph, M. F., Low, H. E., and Zhou, H. (2007). In situ testing for design of pipeline and
646 anchoring systems. Keynote Lecture, *Proc. 6th Int. Offshore Site Investigation and*
647 *Geotechnics Conference: Confronting New Challenges and Sharing Knowledge*,
648 London, 1, 251-262.

649 Stewart, D. P. (1992). "*Lateral loading of piled bridge abutments due to embankment*
650 *construction*." PhD Thesis, The University of Western Australia.

651 Stewart, D. P., and Randolph, M. F. (1991). "A new site investigation tool for the centrifuge."
652 *Proc. Int. Conf. on Centrifuge '91*, pp. 531-538. Rotterdam: Balkema.

- 653 Stewart, D. P., Boyle, R. S., and Randolph, M. F. (1998). "Experience with a new drum
654 centrifuge." *Proc. Int. Conf. on Centrifuge '98*, pp. 35-40. Rotterdam: Balkema.
- 655 Watson, P. G. (1999). "*Performance of skirted foundations for offshore structures.*" PhD
656 Thesis, The University of Western Australia.
- 657 White, D. J., Gaudin, C., Boylan, N., and Zhou, H. (2010). "Interpretation of T-bar
658 penetrometer tests at shallow embedment and in very soft soils." *Canadian*
659 *Geotechnical Journal*, 47(2), 218-229.
- 660 Zhou, M., Hossain, M. S., Hu, Y. and Liu, H. (2013). "Behaviour of ball penetrometer in
661 uniform single- and double-layer clays." *Géotechnique*, 63(8), 682-694.
- 662 Zhou, H., and Randolph, M. F. (2009). "Resistance of full-flow penetrometers in rate-
663 dependent and strain-softening clay." *Géotechnique*, 59(2), 79-86.
- 664 Zhou, H., and Randolph, M. F. (2011). "Effect of shaft on resistance of a ball penetrometer."
665 *Géotechnique*, 61(11), 973-981.

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Table 1. Parameters a, b, c and m for Figure 3

Object	Soil type	a	b	c	m	Reference
T-bar	Kaolin clay	1	2.77	0.30	1.23	Randolph and Hope (2004): with viscous rate effect
T-bar	Kaolin clay	1	2.10	0.30	1.60	Randolph and Hope (2004): without viscous rate effect
T-bar	Kaolin clay	1	2.77	0.17	1.45	Watson and Suemasa (unpublished data, 2009)
Spudcan	Kaolin clay	1	4.45	1.00	1.20	Barbosa-Cruz (2007): based on centrifuge test data
T-bar	Calcareous silt	1	17.00	6.00	1.50	Cassidy (2012)
Spudcan	Calcareous silt	1	17.40	3.50	1.80	Cassidy (2012)
Flat footing & cone	Calcareous silt	1	17.40	0.70	1.00	Finnie and Randolph (1994)

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Table 2. Summary of soil properties

	Kaolin clay (KC) (Stewart 1992; Randolph and Hope 2004)	Calcareous clay (CC) (Watson 1999)	Calcareous silt (CS20) (Cassidy 2012)	Calcareous silt (CS15) (Watson 1999)	
Site	Commercially available	Laminaria; Timor Sea	Gorgon; North-West Shelf, offshore Australia	Gorgon; North-West Shelf, offshore Australia	
Sample	Reconstituted	Reconstituted	Reconstituted	Reconstituted	
Liquid limit, LL (%)	61	82.4	-	-	
Plastic index, I_p (%)	34	30	-	-	
CaCO ₃ content (%)	-	~80	> 95	> 95	
OMYACARB content (%)	-	-	20	15	
Specific gravity, G_s	2.6	No data	2.67	2.76	
Sensitivity, S_t	2~2.8	-	Low	Low	
Consolidation coefficient, c_v (results from Rowe cell test on sample)	2.6 (average) m ² /yr		15 (average over depth of interest) m ² /yr		8 (average) m ² /yr
	σ'_{v0} (kPa)	c_v (m ² /yr)	σ'_{v0} (kPa)	c_v (m ² /yr)	
	0	0	0	0	
	15	1.84	15	15	
	30	2.37	30	25	
	60	3.06	60	35	
	90	3.53	120	49	
	110	3.80	240	82	

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Table 3. Summary of conducted full-spudcan (FS) and T-bar tests in kaolin clay

Event	Test	D_{Spe} = D_{Sp} (m)	D_{Tpe} (m)	D_{Tp} (m)	V_{sb} (m ³)	V_{s} (m ³)	$H_{\text{cavS}}/D_{\text{Spe}}^*$	$H_{\text{ds}}/D_{\text{Spe}}^{\#}$	$H_{\text{dT}}/D_{\text{Tp}}^*$	$H_{\text{dT}}/D_{\text{Spe}}$	γ'_{avg} (kN/m ³)
KC1: NC clay	FS1	3.0	1.13	0.50	0.86	2.20	0.00	0.70	1.50	0.05	7
	FS2	3.0	1.13	0.50	0.86	2.20	0.00	0.70	1.50	0.05	7
	FS3	3.0	1.13	0.50	0.86	2.20	0.00	0.70	1.50	0.05	7
KC2: LOC clay	FS4	2.1	0.79	0.35	0.41	1.04	0.50	1.20	3.48	0.58	7
	FS5	4.2	1.58	0.70	3.26	8.33	0.33	1.03	2.79	0.47	7
	FS6	5.4	1.02	0.45	5.67	14.47	0.30	1.00	3.42	0.29	7
	FS7	12.0	2.26	1.00	54.97	140.36	0.24	0.94	2.33	0.44	7.5
KC3: HOC clay	FS8	6.0	1.13	0.50	6.87	17.54	0.49	1.19	4.33	0.36	7.5

680 *Measured during centrifuge test

681 [#]Calculated using Equation 2682 ^{*}Calculated using Equation 3

Table 4. Summary of conducted full-spudcan, ball (B) and T-bar tests in calcareous clay and silt (Data from Watson 1999; Cassidy 2012)

Event	Test	$D_{\text{Spe}} = D_{\text{Sp}}$ or $D_{\text{Bpe}} = D_{\text{Bp}}$ (m)	D_{Tpe} (m)	D_{Tp} (m)
CC1*: NC clay	B1	1.8	1.36	0.75
CC2: LOC clay	B2	1.8	1.36	0.75
CS(20)1*: NC silt	FS28	6.0	1.13	0.50
	FS29	6.0	1.13	0.50
CS(15)2: NC silt	B3	1.8	1.70	0.75
	B4	1.8	1.70	0.75

*CC: Calcareous clay

*CS: Calcareous silt

688 **Table 5. Summary of normalised penetration rate for full-spudcan, ball and T-bar tests:**
689 **correction factors to be used in Equations 10 and 11**

Event	Test	$D_S =$ D_{Se} (or D_B $= D_{Be}$) (mm)	D_T (mm)	D_{Te} (mm)	v_S or v_B (mm/s)	v_T (mm/s)	c_v (mm/s)	v_S/D_S or v_B/D_B (s ⁻¹)	v_T/D_T (s ⁻¹)	$v_S D_{Se}/c_v$ or $v_B D_{Be}/c_v$	$v_T D_{Te}/c_v$	F_{DS} or F_{DT} or F_{DB}	μ_S or μ_B	μ_T	R_S or R_B	R_T	F_{RSS} or F_{RSB}	F_{RST}
KC1	FS1	30	5	11.3	0.20	1	0.082	0.0067	0.2	72.78	136.87	1	0.12	0.18	3.81	4.40	1.46	1.79
	FS2	30	5	11.3	0.20	1	0.082	0.0067	0.2	72.78	136.87	1	0.12	0.18	3.81	4.40	1.46	1.79
	FS3	30	5	11.3	0.20	1	0.082	0.0067	0.2	72.78	136.87	1	0.12	0.18	3.81	4.40	1.46	1.79
KC2	FS4	30	5	11.3	0.20	1	0.082	0.0067	0.2	72.78	136.87	1	0.12	0.18	3.81	4.40	1.46	1.79
	FS5	30	5	11.3	0.20	1	0.082	0.0067	0.2	72.78	136.87	1	0.12	0.18	3.81	4.40	1.46	1.79
	FS6	60	5	11.3	0.20	1	0.082	0.0033	0.2	145.55	136.87	1	0.12	0.18	3.50	4.40	1.42	1.79
	FS7	60	5	11.3	0.19	1	0.082	0.0031	0.2	137.56	136.87	1	0.12	0.18	3.47	4.40	1.42	1.79
KC3	FS8	60	5	11.3	0.2	1	0.082	0.0033	0.2	145.55	136.87	1	0.12	0.18	3.50	4.40	1.42	1.79
CC1	B1	12	5	11.3	1.00	1	-	0.0833	0.2	-	-	~1	0.05	0.10	4.32	4.40	1.22	1.44
CC2	B2	12	5	11.3	1.00	1	-	0.0833	0.2	-	-	~1	0.05	0.10	4.32	4.40	1.22	1.44
CS(20)1	FS28	60	5	11.3	1.00	1, 3	0.476	0.0167	0.2, 0.6	126.05	71.218	1	0.00	0.03	4.20	4.40, 4.88	1	1.13, 1.15
	FS29	60	5	11.3	2.00	1, 3	0.476	0.0333	0.2, 0.6	252.1	71.218	1	0.00	0.03	4.50	4.40, 4.88	1	1.13, 1.15
CS(15)2	B3, B4	12	5	11.3	1.00	1	0.254	0.0833	0.2	47.244	44.488	1	0.00	0.03	4.32	4.40	1	1.13

690 $N_{Sd,ideal} = 11.3, N_{Td,ideal} = 9.5, N_{Bd,ideal} = 11.5$

691 $(v_T/D_T)_{ref} = 0.5, (v_B/D_B)_{ref} = 0.25, (v_S/D_S)_{ref} = 3.09 \times 10^{-5}/s$

692

693 **Number of Figure: 16**

- 694 **Figure 1.** Single layer seabed profiles: (a) NC, LOC and HOC clay at the Gulf of
695 Mexico (Data from Menzies and Roper 2008), (b) Calcareous very silty
696 clay at North-West Shelf, offshore Australia and calcareous clay at the
697 Gulf of Mexico (Data from Erbrich and Hefer 2002; Menzies and Lopez
698 2011)
- 699 **Figure 2.** Soil failure mechanisms during half-spudcan penetration in single layer
700 clay: (a) Surface heave mechanism; (b) Onset of backflow mechanism; (c)
701 Fully localised flow mechanism
- 702 **Figure 3.** Effect of partial consolidation or drainage
- 703 **Figure 4.** Rate effect in undrained (viscous) region for T-bar and ball in kaolin clay
- 704 **Figure 5.** Combined effect of strain rate and softening on bearing capacity factor
705 (data from Hossain and Randolph 2009b; Zhou and Randolph 2009)
- 706 **Figure 6.** Strain rate and strain softening sensitivity for T-bar and spudcan (Data
707 from Randolph et al. 2007)
- 708 **Figure 7.** Centrifuge model spudcans and T-bar: (a) Spudcan of 60 mm diameter
709 and T-bar of 11.3 mm (equivalent area) diameter; (b) Schematic diagram
710 of load reference point (LRP) and penetration depth for penetrating
711 spudcan and T-bar
- 712 **Figure 8.** Spudcan and T-bar penetration response in normally consolidated kaolin
713 clay (Tests FS1~FS3, Table 3): (a) As a function of normalised tip
714 penetration depth (d/D_{Spe}); (b) As a function of normalised LRP
715 penetration depth (d/D_{Spe})
- 716 **Figure 9.** Ball and T-bar penetration response in normally consolidated and lightly
717 overconsolidated calcareous clay (Tests B1 and B2, Table 4)
- 718 **Figure 10.** Results in normally consolidated calcareous silt: (a) Spudcan and T-bar
719 penetration response (Tests FS28 and FS29, Table 4); (b) Ball and T-bar
720 penetration (and extraction) response (Tests B3 and B4, Table 4)
- 721 **Figure 11.** Spudcan and T-bar penetration response in lightly overconsolidated
722 kaolin clay: (a) Tests FS4~FS6 (Table 3); (b) Test FS7 (Table 3)
- 723 **Figure 12.** Soil flow mechanisms during spudcan and T-bar penetration in single
724 layer clays: (a) Full-spudcan penetration in NC clay with zero strength at
725 the mudline ($s_{um}/\gamma' D_{Spe} = 0$): immediate backflow, $H_{cavS}/D_{Spe} = 0$; (b) Half-
726 spudcan penetration in HOC clay with high strength at the mudline
727 ($s_{um}/\gamma' D_{Spe} = 1.5$): flow towards the surface with minor backflow after
728 $d/D_{Spe} = 1.6$
- 729 **Figure 13.** Spudcan and T-bar penetration response in heavily overconsolidated
730 kaolin clay
- 731 **Figure 14.** Exercise with higher and lower spudcan penetration rate i.e. with effect of
732 partial consolidation and viscous rate effect

- 733 **Figure 15.** Field data from jack-up installation in the Gulf-of-Mexico: (a) Spudcan
734 and (inferred) T-bar penetration response in NC and LOC clay; (b)
735 Spudcan and (inferred) T-bar penetration response in overconsolidated
736 clay
- 737 **Figure 16.** Field data from jack-up installation at Yolla, offshore Australia

(a) NC, LOC and HOC clay at the Gulf of Mexico (Data from Menzies and Roper 2008)

(b) Calcareous very silty clay at North-West Shelf, offshore Australia and calcareous clay at the Gulf of Mexico (Data from Erbrich and Hefer 2002; Menzies and Lopez 2011)

Figure 1. Single layer seabed profiles

Figure 2. Soil failure mechanisms during half-spudcan penetration in single layer clay:
(a) Surface heave mechanism; (b) Onset of backflow mechanism; (c) Fully localised
flow mechanism

Figure 3. Effect of partial consolidation or drainage

Figure 4. Rate effect in undrained (viscous) region for T-bar and ball in kaolin clay

Figure 5. Combined effect of strain rate and softening on bearing capacity factor (data from Hossain and Randolph 2009b; Zhou and Randolph 2009)

Figure 6. Strain rate and strain softening sensitivity for T-bar and spudcan (Data from Randolph et al. 2007)

(a) Spudcan of 60 mm diameter T-bar of 11.3 mm (equivalent area) diameter

(b) Schematic diagram of load reference point (LRP) and penetration depth for penetrating spudcan and T-bar

Figure 7. Centrifuge model spudcans and T-bar

(a) As a function of normalised tip penetration depth (d_t/D_{Spe})

(b) As a function of normalised LRP penetration depth (d/D_{Spe})

Figure 8. Spudcan and T-bar penetration response in normally consolidated kaolin clay (Tests FS1~FS3, Table 3)

Figure 9. Ball and T-bar penetration response in normally consolidated and lightly overconsolidated calcareous clay (Tests B1 and B2, Table 4)

(a) Spudcan and T-bar penetration response (Tests FS28 and FS29, Table 4)

(b) Ball and T-bar penetration (and extraction) response (Tests B3 and B4, Table 4)

Figure 10. Results in normally consolidated calcareous silt

(a) Tests FS4~FS6 (Table 3)

(b) Test FS7 (Table 3)

Figure 11. Spudcan and T-bar penetration response in lightly overconsolidated kaolin clay

(a) Full-spudcan penetration in NC clay with zero strength at the mudline ($s_{um}/\gamma' D_{Spe} = 0$):
immediate backflow, $H_{cavS}/D_{Spe} = 0$

(b) Half-spudcan penetration in HOC clay with high strength at the mudline ($s_{um}/\gamma' D_{Spe} = 1.5$):
flow towards the surface with minor backflow after $d/D_{Spe} = 1.6$

Figure 12. Soil flow mechanisms during spudcan penetration in single layer clays

Figure 13. Spudcan and T-bar penetration response in heavily overconsolidated kaolin clay

Figure 14. Exercise with higher and lower spudcan penetration rate i.e. with effect of partial consolidation and viscous rate effect

(a) Spudcan and (inferred) T-bar penetration response in NC and LOC clay

(b) Spudcan and (inferred) T-bar penetration response in overconsolidated clay

Figure 15. Field data from jack-up installation in the Gulf-of-Mexico

Figure 16. Field data from jack-up installation at Yolla, offshore Australia