Prediction of Spudcan Penetration Resistance Profile in Stiff-Over-Soft Clays

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Abstract

Spudcan punch-through during installation and preloading process is one of the key concerns for the jack-up industry. This incident occurs in layered deposits, with new design approaches for spudcan penetration in sand-over-clay deposits reported recently. This paper reports a novel design approach for spudcan penetration in stiff-over-soft clay deposits. Large deformation finite element (LDFE) analyses were carried out using the Coupled Lagrangian-Eulerian (CEL) approach. The clay was modelled using the extended elastic-perfectly plastic Tresca soil model allowing strain softening and rate dependency of the undrained shear strength. A detailed parametric study was undertaken, varying the strength ratio between bottom and top soil layers, the thickness of the top layer relative to the spudcan diameter, and degree of non-homogeneity of the bottom layer. Existing data from centrifuge model tests were first used to validate the LDFE results, and then the measured and computed datasets were used to develop the formulas in the proposed design approach. The approach accounts for the soil plug in the bottom layer, and the corresponding additional resistance. Where there is the potential for punch-through, the approach provides estimations of the depth and bearing capacity at punch-through, the bearing capacity at the stiff-soft layer interface, and the bearing capacity in the bottom layer. Comparison shows that the punch-through method suggested in ISO standard 19905-1 provides conservative estimate of the bearing capacity at punch-through, with guidelines provided to improve the method.

KEYWORDS: bearing capacity; punch-through; clays; footings/foundations; numerical modelling, offshore engineering

INTRODUCTION

Spudcan Foundation and Punch-through

Most offshore drilling in shallow to moderate water depths is performed from self-elevating jack-up rigs due to their proven flexibility, mobility and cost-effectiveness (CLAROM 1993; Randolph et al. 2005). Jack-ups possess a self-installation capacity. When on site their legs are lowered to the seabed and the footings are then preloaded by pumping seawater into ballast tanks in the hull. This is to ensure that the footings have sufficient reserve capacity in any extreme storm design event.

For the jack-up industry, one of the major concerns related to geotechnical/structural failures during installation is spudcan punch-through. This incident can occur in stratified soil conditions with a surface or interbedded strong layer overlying a weak layer, with rapid leg penetration. Excessive uncontrolled leg plunge may lead to buckling of the leg, effectively decommissioning the platform, or even toppling of the jack-up unit (Aust 1997; Kostelnik et al. 2007; Menzies and Lopez 2011).

Recently, design approaches for spudcan penetration in sand-over-clay deposits have been reported by Teh et al. (2009), Lee et al. (2013a, 2013b), Hu et al. (2014a, 2014b, 2015); and for spudcan penetration in stiff-over-soft clay deposits by Dean (2008). This paper develops a new design approach for spudcan penetration in stiff-over-soft clay deposits with the potential for punch-through (see Figure 1). For jack-up installation in the field, it is critical to know the depth and magnitude of the peak bearing capacity (d_p , P_{peak}) in the top layer and the punch-through distance, h_{p-T} . Comparing P_{peak} with the intended full preload V_p , likelihood and severity of punch-through can be assessed. Even if $V_p \leq P_{peak}$, sufficient safety margin is required to be applied to avoid subsequent failure during e.g. a storm event. For $V_p > P_{peak}$, it is essential to know the severity of potential punch-through in particular h_{p-T} .

68 **ISO Method**

69 For assessing spudcan penetration resistance in stiff clay overlying soft clay, current design
 70 method recommended in the ISO standard 19905-1 (ISO 2012) is based on the following
 71 punching-shear method [originally proposed by Brown and Meyerhof (1969)]

$$\begin{aligned}
 & Q_v = A \left\{ \frac{4T}{D} 0.75 s_{ut} + \text{Min} \left[\left(6 + 1.2 \frac{d_{int}}{D} \right), 9.0 \right] s_{ub} + p'_0 \right\} \\
 & [1] \leq A \left\{ \text{Min} \left[\left(6 + 1.2 \frac{d}{D} \right), 9.0 \right] s_{ut} + p'_0 \right\}
 \end{aligned}$$

73 where Q_v is the gross penetration resistance assuming an open cavity (i.e. the effect of
 74 backfilled soil has not been included in Equation 1), T is the thickness of the top layer
 75 between the base of the advancing spudcan and the initial layer interface, p'_0 is the effective
 76 overburden pressure of soils at the penetration depth d that is measured from the mudline to
 77 the lowest point of spudcan's maximum bearing area, and A is the largest spudcan plan area.
 78 The other terms can be found in Figure 1. No guidelines are given for the punch-through
 79 criterion about the depth of punch-through (d_p) and about selecting the undrained shear
 80 strength s_{ub} for non-homogeneous strength profile.

81 The ISO method was developed based on the results for a surface circular footing on uniform
 82 clays through model tests at $1g$. The base of the soil plug is assumed to be fixed at the initial
 83 stiff-soft layer interface regardless of the spudcan penetration. As such, the soil plug carried
 84 down with the spudcan from the top (stiff) layer into the bottom (soft) layer, and
 85 corresponding contribution to the penetration resistance, are neglected. Additionally, the
 86 strain softening due to remoulding of clay is not explicitly considered, although a factor of
 87 0.75 is applied on intact s_{ut} (see Equation 1) to obtain mobilised shear strength of the top
 88 layer. The need to take full account of softening effect in order to achieve accurate prediction

of spudcan penetration is emphasised by Erbrich (2005), Osborne (2007), Randolph et al. (2007), and Hossain and Randolph (2009a).

Previous Work

To improve the ISO method, a number of investigations have been conducted on the bearing response of foundations in stiff-over-soft clays. Wang and Carter (2002) simulated continuous penetration of a circular footing in two-layer clay deposits, using a large deformation finite element (LDFE) method. Small strain FE analyses of surface footings on uniform-over-uniform clays were undertaken by Edwards and Potts (2004) and Merifield and Nguyen (2006). The results from centrifuge model tests and LDFE analyses for spudcan penetration in stiff-over-soft clays were reported by Hossain and Randolph (2010a, 2010b). Based on the results from LDFE analyses, Hossain and Randolph (2009b) proposed a design approach for predicting the spudcan penetration resistance profile in stiff-over-soft clays. However, the clay was modelled as a non-softening plastic material in the simulation. Therefore, the form of the penetration resistance profile, including the depth and value of peak resistance, was not simulated accurately and the penetration resistance profiles are consistently overestimated.

Edwards and Potts (2004) summarised the results from their small strain FE analyses and proposed a new design approach (Edwards-Potts method) for spudcan penetration in uniform stiff-over-soft clays. For spudcan in the top layer, the contribution to the penetration resistance from the upper stiff layer is considered as a proportion of the difference between the bearing capacities of stiff layer and soft layer. The value of the proportion can be estimated from the proposed formula as a function of the strength ratio between clay layers and the thickness ratio of the stiff clay layer. For spudcan penetration in the bottom layer, the conventional single layer bearing capacity formula by Skempton (1951) is recommended.

Based on the centrifuge test data of spudcan penetration in uniform-over-uniform clay reported by Hossain and Randolph (2010a), Dean (2011) proposed an improvement of the ISO method (Dean method) in the discussion with Hossain and Randolph (2011). In the Dean method, the thickness of the soil plug, equal to the top layer thickness, is assumed to be unchanged during the spudcan penetration so that the bearing resistance from the periphery and base of the soil plug in the underlying soft layer can be calculated. For the calculation of spudcan penetration resistance in the top layer, adjustment coefficients were back-calculated for the terms of shear resistance and end bearing capacity. No recommendations were given for the calculation of the bottom layer. Hossain and Randolph (2011) supplemented the data with those from additional centrifuge tests and LDFE analyses for spudcan penetration in uniform-over-non-uniform clay, with slightly different adjustment coefficients obtained from the best fit.

Existing Database and Objective of Present Study

The centrifuge tests reported by Hossain and Randolph (2010a) for spudcan penetration in stiff-over-soft clays are assembled in Table 1. The shear strengths in these tests were measured using T-bar penetrometer test and the calculation framework at that time with a deep bearing capacity factor of 10.5. However, recent studies have highlighted two issues for characterising stiff-over-soft clay deposits with T-bar penetrometer (White et al. 2010; Lee et al. 2013a; Zhou et al. 2013): (a) for thin top layers relative to the penetrometer diameter, the full (i.e. stable) penetration resistance of that layer may not be mobilised; and (b) in the underlying soft layer, a soil plug may be brought down by the penetrometer from the top layer and hence the mobilised resistance may be higher compared with the actual resistance of that layer. Adjustments are therefore required. This is particularly critical for the T-bar penetrometer of 0.5 m (prototype) diameter used in centrifuge tests. As such, the strength

values are corrected through trial and error by simulating T-bar penetration in stiff-over-soft clays, which is discussed in the section on ‘Simulation of centrifuge tests’.

A numerical parametric study (Groups II~IV, Table 2) was carried out to complement the centrifuge test data. To overcome the drawbacks that the clay was modelled as idealised rate-independent, non-softening material in the previous numerical analyses (Wang and Carter 2002; Hossain and Randolph 2010b), the effects of strain softening and rate dependency of the undrained shear strength of clay were incorporated.

Based on the database (Tables 1 and 2), this paper aims to (i) present an extensive LDFE parametric study for spudcan penetration in stiff-over-soft clay accounting for strain softening and rate dependency; and (ii) propose a design approach to predict the depth and magnitude of the peak penetration resistance in the top layer, the resistance at the layer interface, and the penetration resistance profile in the bottom layer.

NUMERICAL MODEL

Geometry and Parameters

This study has considered a circular spudcan of diameter D , penetrating into a two-layer clay deposit as illustrated schematically in Figure 1, where the top stiff layer with intact uniform undrained shear strength s_{ut} , effective unit weight γ'_t , and thickness t is underlain by the bottom soft layer of non-uniform intact undrained shear strength $s_{ub} = s_{ubs} + k(z - t)$, and effective unit weight γ'_b . s_{ubs} is the intact undrained shear strength of the bottom soft layer at the layer interface. $s_{ub0} = s_{ubs} + \text{Max}[k(d - t), 0]$ is the intact undrained shear strength at the spudcan base level d . Analyses were undertaken for a spudcan with a 13° shallow conical underside profile (included angle of 154°) and a 76° protruding spigot of height $0.14D$. The

spudcan shape is similar to the spudcans of the Marathon LeTourneau Design, Class 82-SDC jack-up rig, as illustrated by Menzies and Roper (2008).

A series of parametric analyses (Groups II~IV, Table 2) were performed simulating continuous penetration of spudcan from the seabed. The thickness of the top layer, t , was varied relative to the spudcan diameter as $0.25D \sim 1D$, with nominally infinite thickness of the bottom layer. A thickness ratio of $t/D > 1$ was not considered in the numerical parametric study. The ratio at the interface between the bottom and top layers, $S_{\text{ubs}}/S_{\text{ut}}$, was varied as $0.25 \sim 0.75$. For convenience, the effective unit weight of the deposit was considered to be constant and was taken as $\gamma' = \gamma'_t = \gamma'_b = 8 \text{ kN/m}^3$. The normalised strength of the bottom layer at the interface was $S_{\text{ubs}}/\gamma'_b D = 0.31$ (to reduce the effect of less influential factor, while keeping the value within the range of practical interest; Hossain and Randolph 2010b), with the degree of non-homogeneity $kD/S_{\text{ubs}} = 0 \sim 3$.

Analysis Details and Model Set-up

The LDFE analyses in this study were performed using the Coupled Eulerian-Lagrangian (CEL) approach in the commercial FE package Abaqus/Explicit (DSS 2010). The CEL approach has already been employed to simulate spudcan penetration in multi-layer clay deposits, as discussed in detail by Zheng et al. (2014, 2015). Other applications of this technique include cone and strip footing penetration in cohesive soils (Qiu and Grabe 2011), spudcan penetration in single- and two-layer clays (Tho et al. 2012), and spudcan penetration in sand overlying clay (Qiu and Henke 2011; Qiu and Grabe 2012; Hu et al. 2014b).

Currently, only three-dimensional elements are available in CEL analyses. For all analyses in this study, only a quarter sector of the domain was involved accounting for the inherent symmetry, as shown in Figure 2. The soil domain was chosen as $5D$ in horizontal and $8D$ in depth to avoid boundary effect. The spudcan was modelled as a rigid Lagrangian body and the

soil-spudcan interface as fully rough to ensure soil plug trapped at the base of the advancing spudcan. Note, for penetration of an object in stiff-over-soft clay deposits i.e. penetration of an object with a trapped soil plug underneath, the effect of object base roughness was found to be minimal (Griffiths 1982; Merifield et al. 1999; Merifield & Nguyen 2006; Hossain and Randolph 2010b). The soil was discretised using Eulerian elements of type EC3D8R. The Eulerian mesh included the original soil domain and an overlying layer initially filled with ‘void’ material to accommodate the soil heave that resulted from the spudcan penetration. A cuboid of fine mesh was created along the trajectory of the spudcan, with a constant element size of $0.025D$, which is consistent with the element size adopted by Hu et al. (2014b) and Zheng et al. (2014, 2015).

Constitutive Law and Material Properties

The clay was modelled as an elastic-perfectly plastic material obeying the Tresca yield criterion, but extending to capture strain rate and strain softening effects. The undrained shear strength at each integration point was modified at the beginning of each time step, following Einav and Randolph (2005) as

$$[2] \quad s_{uc} = \left[1 + \mu \log \left(\frac{\text{Max}(|\dot{\gamma}|, \dot{\gamma}_{ref})}{\dot{\gamma}_{ref}} \right) \right] \left[\delta_{rem} + (1 - \delta_{rem}) e^{-3\xi/\xi_{95}} \right] s_u$$

where s_{uc} is the current undrained shear strength considering rate dependency and strain softening, and s_u is the intact undrained shear strength measured at the reference shear strain rate prior to any softening.

The first bracketed term augments the strength according to the maximum shear strain rate $\dot{\gamma}$ relative to a reference value, $\dot{\gamma}_{ref}$, which was selected for the parametric study so that the normalised penetration rate $v/(D\dot{\gamma}_{ref})$ of 11.11 lies within the range of practical interest (4~20;

Hossain and Randolph 2009a; InSafeJIP 2011; Hossain et al. 2014). It should be noted that typical values of spudcan penetration rates in the field may be estimated to lie between 0.4 to 4 m/h, and reference shear strain rate in triaxial test ranges from 1%/h to 4%/h. The corresponding influence on spudcan penetration resistance has been discussed by Hossain and Randolph (2009a). The augment of shear strength follows a logarithmic law with rate parameter μ taken as 0.1 for circular spudcan foundations (Low et al. 2008).

The second part of Equation 2 models the degradation of strength according to an exponential function of accumulated absolute plastic shear strain, ξ , from the intact condition to a fully remoulded ratio, δ_{rem} ($= 1/S_t$, i.e. inverse of sensitivity S_t). The parameter, ξ_{95} , represents the cumulative plastic shear strain required for 95% remoulding. A typical value of $\xi_{95} = 12$, within the range of 10~25 (i.e. 1000~2500% shear strain) suggested by Randolph (2004) and Einav and Randolph (2005) for normal clays with sensitivity $S_t = 2\sim 5$, was used. The same strain rate and strain softening parameters were adopted throughout the soil profile. CEL analyses using the combination of the adopted typical values have been carried out for some centrifuge tests and reported case histories, providing reasonable agreement with the measured data (Hossain et al. 2014; Zheng et al. 2013, 2015).

A uniform stiffness ratio of $E/s_{uc} = 200$ (where E is the Young's modulus) was taken throughout the clay profile. The ratio is within the range commonly adopted for soft clays, but the precise value has negligible effect on the spudcan penetration resistance [see Figure 10 of Tho et al. (2012)]. This is because the clay around the penetrating spudcan predominantly undergoes significant plastic deformation, and hence the bearing capacity is hardly affected by the elastic parameter. Considering the large diameter and relatively fast penetration of spudcans in the field, all the analyses simulated undrained conditions and adopted a Poisson's ratio of 0.49. The geostatic stress conditions were modelled with coefficient of lateral earth

pressure of 1, as the stable penetration resistance was found to be almost unaffected by the coefficient (Zhou and Randolph 2009).

SIMULATION OF CENTRIFUGE TESTS

As discussed previously, the undrained shear strengths reported by Hossain and Randolph (2010a) for the centrifuge tests assembled in Table 1 were re-interpreted in the light of improved understanding about the effect of underlying soft layer on T-bar penetration resistance. Numerical analyses were first undertaken simulating T-bar penetration for centrifuge tests in Table 1 in an effort to explore the actual values of undrained shear strengths. The soil-T-bar interface was modelled as partially rough with the limiting shear stress equal to δ_{remSut} . The accuracy of the numerical model has already been verified by Zheng et al. (2013, 2014, 2015) through validation against centrifuge tests of spudcan penetration in three-layer clays. For each test, the corrected value of undrained shear strength was obtained through trial and error. Several attempts were made to get the corrected value until satisfactory agreement was achieved between the numerical and experimental T-bar penetration resistance profiles. In each attempt, the undrained shear strength was corrected in the light of the error in the last attempt. The computed and measured T-bar penetration responses for tests E2UNU-I-T 3 and E2UNU-II-T 5 are presented in Figure 3 in terms of total bearing pressure q_u as a function of the penetration depth of the T-bar invert. In numerical simulations, two sets of undrained shear strength profiles were used, including one suggested by Hossain and Randolph (2010a) and the corrected one.

Figure 3 indicates that a thickness of $t = 4.5$ m relative to the T-bar diameter of 0.5 m is not thick enough to establish the full penetration resistance in the top layer due to the influence of the bottom soft layer. For the T-bar penetration in the bottom layer, the difference between

numerical results and centrifuge test data is believed to be caused by the stiff soil plug brought down by the T-bar from the top layer. Physical evidence of trapping stronger soil at the base of the advancing T-bar or ball penetrometer was reported by Lee (2009) and Wang et al. (2016, personal communications), showing that a plug from the upper strong (sand or stiff clay) layer was brought down by the penetrometer into the lower soft clay layer. The T-bar bearing pressure profiles from numerical analyses using corrected undrained shear strengths, by contrast, agree well with the centrifuge test data. The apparent discrepancy at shallow penetration depths (prior to the peak) between the measured and computed profiles even using the corrected shear strengths is believed to be due to the following fact. In centrifuge tests, preconsolidated soil samples started to swell immediately after taking off the consolidation pressure. Absorption of water and softening of clay near the free surface occurred during centrifuge spinning (Hossain and Randolph 2010a), leading to lower strength at shallow depths. In LDFE analyses, a uniform strength was considered for the top layer. This simplification did not influence corresponding spudcan penetration resistance, as can be seen in Figure 4.

Spudcan penetration analyses were then performed for tests E2UNU-I-T 3 and E2UNU-II-T 5 using the corrected undrained shear strengths. Figure 4 compares the experimental and numerical results. The computed penetration resistance profiles agree reasonably well with the measured data, confirming the correctness of using the corrected values of undrained shear strength. Therefore, for centrifuge test data, the corrected undrained shear strengths were used to propose the new design approach. All the corrected undrained shear strengths are listed in Table 1.

RESULTS AND DISCUSSION

The load-penetration responses are presented in terms of the normalised net bearing pressure, $q_{\text{net}}/s_{\text{ub}0}$, as a function of the normalised penetration depth, $(d - t)/D$, with q_{net} calculated as

$$[3] \quad q_{\text{net}} = \frac{P}{A} - \gamma' \frac{V}{A}$$

where P is the total penetration resistance, and V is the volume of the embedded spudcan including shaft.

The soil failure mechanisms and hence the penetration resistance profiles of spudcan foundation on stiff-over-soft clays are affected by a number of factors, including the strength ratio $s_{\text{ubs}}/s_{\text{ut}}$, the thickness of the top layer relative to the spudcan diameter t/D , and the strength non-homogeneity factor of the bottom layer kD/s_{ubs} . The effects of these factors on the depth d_p and value q_{peak} of the peak resistance in the top layer and the bearing capacity factors in the bottom layer are briefly discussed below. This will lead to the development of the new design approach.

Effect of Strength Ratio $s_{\text{ubs}}/s_{\text{ut}}$

To investigate the effect of the strength ratio $s_{\text{ubs}}/s_{\text{ut}}$ on the penetration resistance profile, Figure 5 is plotted for strength ratio $s_{\text{ubs}}/s_{\text{ut}} = 0.25, 0.3, 0.4, 0.5$ and 0.75 with $t/D = 0.75$, $s_{\text{ubs}}/\gamma'D = 0.31$, and $kD/s_{\text{ubs}} = 0.5$ (Group II, Table 2). It can be seen that the normalised distance of peak resistance from the layer interface, $(d-t)/D$, changes from -0.58 to -0.03 or the normalised depth from the mudline, d_p/D , increases from 0.17 to 0.72 with increasing $s_{\text{ubs}}/s_{\text{ut}}$ from 0.25 to 0.75 .

The load-penetration response in the bottom layer is also affected by $s_{\text{ubs}}/s_{\text{ut}}$. The normalised resistance close to the interface is higher for a lower strength ratio. This discrepancy

diminishes gradually as the spudcan penetrates deeper. This is caused by the soil plug carried down by the advancing spudcan from the stiff layer and corresponding additional resistance. A lower strength ratio enables a thicker soil plug to be forced down (see insets in Figure 5). This is because, while the spudcan penetrates in the top layer, lower strength ratio attracts penetration mechanism into the bottom soft layer and allows for triggering the punch-through failure earlier or at a shallow penetration depth (see Figure 5), leading to a thicker and stronger soil plug trapped at the base of the advancing spudcan. The soil plug height diminishes gradually with the initiation of soil backflow and the increasing strength in the bottom layer.

Effect of Thickness Ratio t/D

The penetration resistance profiles from analyses of $t/D = 0.25, 0.5, 0.75$ and 1.0 are plotted in Figures 6a and 6b for $S_{\text{subs}}/S_{\text{ut}} = 0.25$ and 0.5 , respectively, with $S_{\text{subs}}/\gamma'D = 0.31$ and $kD/S_{\text{subs}} = 0$ (Group III, Table 2). As t/D increases, the normalised peak resistance depth d_p/D becomes deeper, which is more profound for higher $S_{\text{subs}}/S_{\text{ut}}$. For $S_{\text{subs}}/S_{\text{ut}} = 0.25$, d_p/D increases from 0.12 to 0.17 (Figure 6a), whereas from 0.22 to 0.48 for $S_{\text{subs}}/S_{\text{ut}} = 0.5$ (Figure 6b).

In the bottom layer, the effect of the soil plug is more profound for $S_{\text{subs}}/S_{\text{ut}} = 0.25$, with thicker stiff layer leading to a higher normalised penetration resistance. In contrast, for higher strength ratio of $S_{\text{subs}}/S_{\text{ut}} = 0.5$, all the profiles form a unique line after $(d - t)/D = \sim 0.6$, with $N_{\text{cd}} = q_{\text{net}}/S_{\text{ub0}} = \sim 11.4$.

Effect of Strength Non-homogeneity kD/S_{subs}

The effect of soil strength non-homogeneity, indicated by kD/S_{subs} , on the bearing response of spudcan is specifically examined through Figures 7a and 7b. The penetration resistance profiles are from analyses for $kD/S_{\text{subs}} = 0, 0.25, 0.5$ and 3.0 , with identical $S_{\text{subs}}/\gamma'D = 0.31$ and

$s_{\text{Subs}}/s_{\text{Sut}} = 0.25$, but for thickness ratios of $t/D = 0.5$ and 1.0 , respectively (Group IV, Table 2).

It can be seen that the depth of peak resistance d_p/D increases as kD/s_{Subs} increases.

The effect of kD/s_{Subs} on the bearing response in the bottom layer is significant particularly close to the layer interface. For high non-homogeneity factor of $kD/s_{\text{Subs}} = 3.0$, the normalised penetration resistance is higher at the layer interface, but it reduces rapidly with penetration depth and finally stabilises at a depth of $(d - t)/D = \sim 1.0$. In contrast, for non-homogeneity of $kD/s_{\text{Subs}} = 0$, the normalised penetration resistance is lower over the early penetration in the bottom layer. However, the profile reduces at a lower rate, due to the presence of the soil plug that diminishes slower (see insets in Figure 7), leading to higher normalised penetration resistance over deep penetration.

Effect of Sensitivity S_t

The above parametric study was undertaken for a consistent value of sensitivity, $S_t = 2.8$ for both top and bottom layers. In order to examine the effect of sensitivity on the penetration resistance, additional analyses were carried out with $S_t = 1$ (rate-dependent soil without strain softening) and 5 for a representative case ($s_{\text{Subs}}/s_{\text{Sut}} = 0.5$, $t/D = 1.0$, $kD/s_{\text{Subs}} = 0.5$ and $s_{\text{Subs}}/\gamma'D = 0.31$, Group V, Table 2).

The penetration resistance profiles for different sensitivities are plotted in Figure 8, with all the other soil parameters in Equation 2 unchanged. As S_t increases from 1 to the practical value of 2.8, the depth and value of the peak resistance is considerably affected. The values of d_p/D and $q_{\text{peak}}/s_{\text{Subs}}$ decrease from 1.0 to 0.58 and from 19.2 to 16.2, respectively, whereas the penetration resistance in the bottom layer is reduced by $\sim 25\%$. This is close to the suggestion by Menzies and Roper (2008) and Hossain et al. (2014) who recommended a 20% reduction to be applied to the numerical modelling results of spudcan penetration in single-layer non-softening and rate-independent clay. With further increase of S_t from 2.8 to 5, the depth and

value of the peak resistance decrease marginally, while the deep penetration resistance is reduced further by ~10%. Note, the presented results are from analyses considering typical rate parameter $\mu = 0.1$ and softening parameter $\xi_{95} = 12$ for circular spudcan foundations (Low et al. 2008; Hossain and Randolph 2009a). An equivalent normalised resistance or bearing capacity factor for other combinations of μ and ξ_{95} may be obtained using Figure 8 of Hossain and Randolph (2009a). Through sensitivity analyses, Hossain and Randolph (2009a) quantified the influences of μ and ξ_{95} .

NEW DESIGN APPROACH

Based on the results from centrifuge tests reported by Hossain and Randolph (2010a), as tabulated in Table 1, and LDFE analyses in this study (Groups II~IV, Table 2), design formulas are developed for predicting the penetration resistance profile of spudcan in stiff-over-soft clay deposit. The new design approach aims to predict the position and magnitude of the peak resistance (d_p , q_{peak}) in the top layer, the bearing capacity at the stiff-soft layer interface (d_{int} , q_{int}), and the deep bearing capacity factor N_{cd} . For the prediction of q_{peak} , two design methods are proposed, including the semi-empirical method and the improved ISO method.

Peak Resistance

Depth of peak resistance d_p

According to the previous discussion, the depth of peak resistance relative to the spudcan diameter, d_p/D , varies as a function of normalised parameters, S_{ubs}/S_{ut} , t/D and kD/S_{ubs} . The trend is shown in Figure 9 plotting the values from centrifuge tests and parametric study, which can be expressed as

368 [4]
$$\frac{d_p}{D} = 1.3 \left(\frac{s_{\text{ubs}}}{s_{\text{ut}}} \right)^{1.5} \left(\frac{t}{D} \right) \left(1 + \frac{kD}{s_{\text{ubs}}} \right)^{0.5} \leq \frac{t}{D}$$

369 Once d_p/D is determined, the corresponding penetration resistance at the peak can be
 370 calculated using either the semi-empirical method or the improved ISO method, as introduced
 371 below. Note, for layered clay deposits with uniform bottom (soft) layer ($k = 0$), Equation 4
 372 indicates that d_p/t depends only on strength ratio $s_{\text{ubs}}/s_{\text{ut}}$, i.e. independent of spudcan diameter,
 373 which is confirmed by Figure 6.

374 *Magnitude of peak resistance q_{peak}*

375 *Semi-empirical method*

376 Figure 10a shows the relation between the normalised net peak resistance $q_{\text{peak}}/s_{\text{ubs}}$ plotted on
 377 the vertical axis, and a correlation expression plotted on the horizontal axis. The values
 378 measured in centrifuge tests are in good agreement with the LDFE results, and interestingly
 379 all the data show a unique trend, which may be expressed as

380 [5]
$$\frac{q_{\text{peak}}}{s_{\text{ubs}}} = 6.35 + 5 \left[\left(\frac{s_{\text{ubs}}}{s_{\text{ut}}} \right)^{-1} \left(\frac{t}{D} \right)^{0.75} \left(1 + \frac{kD}{s_{\text{ubs}}} \right)^{0.5} \right]^{0.77} \geq \frac{q_{\text{nets}}}{s_{\text{ubs}}}$$

381 where q_{nets} is the net penetration resistance at the top of the stiff layer calculated using the ISO
 382 method (Equation 1). The relationship between Equation 5 and values of $q_{\text{peak}}/s_{\text{ubs}}$ obtained
 383 from centrifuge tests and numerical analyses is shown in Figure 10a, while Figure 10b shows
 384 that the design formula predicts the peak resistance mostly within an error of 10%.

385 *Improved ISO method*

386 The ISO method for punch-through is given in Equation 1. The corresponding punch-through
 387 model is shown in Figure 11, with the base of stiff soil plug fixed at the stiff-soft layer

interface and hence without considering the soil plug in the soft clay. The 1st term of Equation 1 represents the shear resistance along the shear planes in the stiff layer, while the 2nd term calculates the end bearing capacity for a fictitious footing at the layer interface assuming a general shear failure.

The improvement of the ISO method is proposed as follows. The depth of punch-through is assumed as identical to d_p obtained from Equation 4. The penetration resistance at peak, $Q_{v,peak}$, is then calculated for all cases (Tables 1 and 2) using Equation 1 with $d = d_p$ but adding a plug (of height T') in the bottom layer and shifting the fictitious footing position to the base of the added plug, as illustrated in Figure 11. A factor of 0.75 is applied to the shear resistance around the periphery of the cylindrical soil plug in the bottom layer to be consistent with the calculation in the top layer using the ISO method (see Equation 1). Adding the bearing resistance provided by the soil plug in the soft layer ($4.2T'_{Subs}/D$: $3T'_{Subs}/D$ from the shear resistance around the plug and $1.2T'_{Subs}/D$ from the end bearing capacity; see Figure 11) to Equation 1, the improved punch-through criterion for spudcan penetration in uniform stiff-over-soft clay deposit ($s_{ub} = s_{ubs}$) can be expressed as

$$[6] \quad Q_{v,peak} = A \left\{ 3 \frac{T}{D} s_{ut} + \text{Min} \left[6 \left(1 + 0.2 \frac{d_{int}}{D} \right), 9.0 \right] s_{ub} + p'_0 + 4.2 \frac{T'}{D} s_{ubs} \right\}$$

To keep a simple form of the formula, Equation 6 is also used to predict the peak resistance for spudcan penetration in uniform-over-non-uniform deposit (i.e. $k > 0$), taking s_{ub} in Equation 6 as the average strength over the depth of $D/2$ below the layer interface. Calibration of the calculated values of $Q_{v,peak}$ against the measured and computed data provides the values for the equivalent soil plug thickness T' . All the normalised values of T'/t are plotted in Figure 12a with the approximation given as

$$[7] \quad \frac{T'}{t} = 0.53 \left(\frac{s_{\text{ubs}}}{s_{\text{ut}}} \right)^{-0.5} \left(\frac{t}{D} \right)^{-1} \left[3 - \left(1.5 - \frac{kD}{s_{\text{ubs}}} \right)^2 \right]^{0.5 \frac{t}{D}} \quad \text{for } kD/s_{\text{ubs}} \leq 3$$

It can be seen in Figure 12b that the proposed method predicts q_{peak} within an error of $\pm 10\%$ for all cases except one centrifuge test with low peak resistance.

The third bracketed term in Equation 7 increases with increasing bottom layer non-homogeneity kD/s_{ubs} for $kD/s_{\text{ubs}} \leq 1.5$. This is because higher shear resistance and end bearing capacity are mobilised by the stiff soil plug in the lower layer non-uniform clay for higher kD/s_{ubs} , leading to a higher value of equivalent soil plug thickness T' which implicitly includes these effects (as s_{ubs} is used in Equation 6). The term starts to decrease with increasing kD/s_{ubs} for $kD/s_{\text{ubs}} > 1.5$. This is because higher kD/s_{ubs} reduces the potential for punch-through significantly, and hence the thickness of the stiff soil plug carried down into the bottom layer. The higher non-homogeneity or shear strength gradient would also lead to an overestimate for the empirical value of s_{ub} in Equation 6, which is taken as the average shear strength over a depth of $D/2$ below the layer interface. These effects are found to be coupled with the thickness ratio t/D of the stiff clay layer. Therefore, higher value of kD/s_{ubs} may lead to an unsafe prediction of penetration resistance using Equation 7 and it is suggested that the design formula is applicable to the range of $kD/s_{\text{ubs}} \leq 3$ explored in this study (see Table 2).

The improved ISO method explicitly considers the effect of the soil plug pushed into the underlying layer in the calculation of the bearing capacity at punch-through. The method modifies the original formula without changing the framework of bearing capacity calculation recommended by ISO standard 19905-1 (ISO 2012).

431 **Resistance at Layer Interface q_{int}**

432 The severity of punch-through failure or the degree of reduction of bearing capacity can be
 433 indicated by connecting the points of spudcan resistances at punch-through (d_p , q_{peak}) and at
 434 upper-lower layer interface (d_{int} , q_{int}). As such, the normalised bearing capacities, q_{int}/s_{ubs} , at
 435 the upper-lower layer interface, from centrifuge tests and numerical analyses are plotted in
 436 Figure 13, which is best fitted by

$$437 \quad [8] \quad \frac{q_{int}}{s_{ubs}} = 10.2 \left[\left(\frac{s_{ubs}}{s_{ut}} \right)^{-0.5} \left(\frac{t}{D} \right)^{0.25} \left(1 + \frac{kD}{s_{ubs}} \right)^{0.25} \right]^{0.85} \leq \frac{q_{peak}}{s_{ubs}}$$

438 **Deep Bearing Capacity Factor N_{cd}**

439 For spudcan penetration resistance in the bottom layer, a simplified single layer approach is
 440 used, which implicitly incorporates the effect of the soil plug and corresponding additional
 441 resistance in the deep bearing capacity factor N_{cd} . To explore the value of N_{cd} , all profiles of
 442 q_{net}/s_{ub0} as a function of $(d - t)/D$ from numerical analyses and centrifuge tests are plotted in
 443 Figure 14 with a lower bound of 9.8 and an upper bound of 15.5. The corresponding stabilised
 444 values of N_{cd} are plotted in Figure 15, with the best fit linear line expressed as

$$445 \quad [9] \quad N_{cd} = 9.8 + 1.3 \left(\frac{s_{ubs}}{s_{ut}} \right)^{-1} \text{Min} \left[\left(\frac{t}{D} \right)^{0.5}, 1.0 \right] \left(1 + \frac{kD}{s_{ubs}} \right)^{-1}$$

446 For the range of soil properties and layer geometries explored in this study, Equation 9
 447 predicts the value of N_{cd} mostly within an error of ± 1.0 . It should be noted that, in Equations
 448 4, 5, 7, 8 and 9, exponents for (s_{ubs}/s_{ut}) , (t/D) , and $(1 + kD/s_{ubs})$ are different according to their
 449 degree and type (direct or inverse) of influence.

Summary of the Proposed Assessment Procedure

The assessment procedure for practical application is summarised as follows. The first step is to predict the depth of peak resistance d_p in the top layer using Equation 4, and corresponding peak resistance q_{peak} using Equation 5 according to the semi-empirical method or $Q_{v,peak}$ using Equation 6 combined with Equation 7 according to the improved ISO method. The total vertical reaction P_{peak} at $d = d_p$ then can be calculated by adding the buoyancy $\gamma'V$ to Aq_{peak} , or by adding the buoyancy $\gamma'V$ to $Q_{v,peak}$ and then deducting the weight of the backfill soil above the spudcan. Once P_{peak} is determined, the potential for punch-through is assessed by comparing P_{peak} with the intended preload V_p . If $P_{peak} \leq V_p$, punch-through failure would not occur and the spudcan would rest at a depth $\leq d_p$ (although in practice, a safety factor should be applied e.g. $V_p \leq \sim 0.75P_{peak}$). Otherwise, estimate the penetration resistances at the layer interface, $(Aq_{int} + \gamma'V)$, and in the bottom layer, $(AN_{cdSub0} + \gamma'V)$, with q_{int} and N_{cd} calculated using Equations 8 and 9, respectively, and determine the punch-through distance h_{p-T} . The final spudcan-resting depth is determined by comparing $(AN_{cdSub0} + \gamma'V)$ with V_p .

If a complete penetration resistance profile is required, straight lines can be used to connect the penetration resistances at the surface of the stiff layer, at $d = d_p$ and at $d = d_{int}$, and the penetration resistance profile in the bottom layer (see Figure 16). The bearing capacity at the surface of the stiff layer can be calculated using Equation 1.

The proposed design approach generally applies to stiff-over-soft clay deposits with a practical range of soil parameters and sensitivity of around 2.8. For spudcan penetration in highly sensitive clay deposits, the depth and value of the peak resistance in the top layer and the penetration resistance in the bottom layer will be lower.

APPLICATION

As shown in Figure 16, the proposed methods are used to predict the data from centrifuge tests E1UU-I-T 3 and E2UNU-II-T 5 in Table 1. The predictions from the ISO, Edwards-Potts and Dean methods are also included in the figure for comparison. As the ISO and Edwards-Potts methods are proposed for uniform clays, the strength of the bottom layer non-uniform clay is taken as the average over the depth of $D/2$ below the layer interface. For the application of the Dean method, adjustment coefficients from Hossain and Randolph (2011) for uniform-over-non-uniform clay profiles are used.

Figure 16 shows that the depth of peak resistance d_p predicted by Equation 4 is with an error $< 0.2D$, while the other methods indicate the peak resistance at the top surface of the stiff layer. Compared with the centrifuge test data, the corresponding peak resistances from the semi-empirical method and improved ISO method are, respectively, 5.6 and 5.8% higher for test E1UU-I-T 3, and 0.5% lower and 2.3% higher for test E2UNU-II-T 5. In contrast, those from the ISO, Edwards-Potts and Dean methods are, respectively, 13.5 and 13.4% lower and 0.3% higher than the centrifuge test data for test E1UU-I-T 3, and 21.1, 0.7 and 6.6% lower for test E2UNU-II-T 5. The magnitude of the peak resistance is better predicted by the proposed methods due to the fact that the methods were proposed based on the experimental and numerical studies of continuous spudcan penetration, and the contribution from the soil plug in the bottom layer to the bearing capacity is considered.

For penetration resistance in the bottom layer, the bearing capacity factors for circular footing on single-layer uniform clay reported by Skempton (1951) are recommended by ISO (2012) without considering the effect of the trapped soil plug. In the calculation for non-uniform clay, the average strength over a depth of $D/2$ below the spudcan base is considered. It can be seen in Figure 16 that for test E1UU-I-T 3, the deep penetration resistance predicted by the ISO

method is approximately 20.5% lower than the centrifuge test, while the prediction from the proposed approach is very close to the centrifuge test data. For test E2UNU-II-T 5, the proposed approach agrees well with the centrifuge test data with an underestimate of about 4%, while the penetration resistance profile from the ISO method is overall ~15% lower.

For intended preload that exceeds the peak resistance, the punch-through distances from centrifuge tests are $h_{P-T} = \sim 0.31D$ and $\sim 1.55D$ for tests E1UU-I-T 3 and E2UNU-II-T 5, respectively. The ISO, Edwards-Potts and Dean methods that predict the bearing capacity profile in the top layer, combined with the Skempton's (1951) method for the calculation of the bottom layer, result in punch-through distances $h_{P-T} = \sim 0.94D$, $\sim 0.94D$ and $\sim 1.56D$ respectively for test E1UU-I-T 3, and $h_{P-T} = \sim 1.68D$, $\sim 2.24D$ and $\sim 2.08D$ for test E2UNU-II-T 5. In contrast, the punch-through distances indicated by the proposed semi-empirical method and improved ISO method are both $\sim 0.29D$ for test E1UU-I-T 3, and respectively $\sim 1.76D$ and $\sim 1.83D$ for test E2UNU-II-T 5.

CONCLUSIONS

This paper has reported LDFE analyses of spudcan penetration in stiff-over-soft clay deposits with the effect of strain softening and rate dependency of the undrained shear strength accounted for. The existing data from centrifuge model tests were accumulated, with the effect of the underlying soft clay layer on the T-bar penetration resistance not understood at that stage (i.e. the effect of layering neglected) now rectified. The re-calibration exercise of strengths measured in centrifuge model tests using a T-bar penetrometer (along with a deep bearing capacity factor) in stiff-over-soft clays has suggested that adjustments are required to apply to take into account the effect of layering unless a recent approach is adopted assessing the strength of each layer separately through testing on single layer deposit.

Based on the centrifuge test data and LDFE results, a simple design approach was proposed for predicting spudcan penetration in stiff-over-soft clay deposits. This approach provides estimates of (a) the peak penetration resistance and its depth in the top stiff clay layer, (b) the resistance at the stiff-soft layer interface, and (c) the penetration resistance profile in the bottom soft layer. An improvement to the design formula suggested by ISO for punch-through was proposed to predict the peak penetration resistance in the stiff layer. Comparison between the predictions using the ISO method, recently developed methods and the proposed approach, and measured data from centrifuge tests demonstrated the improvement by the proposed approach. For assessing spudcan penetration resistance in stiff-over-soft clay as part of a multi-layer soil profile, it is recommended that the proposed approach should be used with caution. This is because the presence of another layer above or below the stiff-over-soft clay will affect the mobilised soil failure mechanisms and the soil plug thickness trapped at the base of the advancing spudcan, and hence the penetration resistance profile.

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673 LIST OF SYMBOLS

674	A	spudcan plan area at largest section
675	D	spudcan diameter at largest section
676	d	penetration depth of spudcan base (lowest point at largest section)
677	d_{int}	depth of stiff-soft layer interface
678	d_p	depth of peak penetration resistance in top layer soil
679	E	Young's modulus
680	H_{cav}	open cavity depth after spudcan installation
681	h_{p-T}	punch-through distance
682	k	rate of increase of shear strength within bottom layer
683	N_{cd}	deep bearing capacity factor in the bottom layer soil
684	P	total penetration resistance of spudcan
685	p'_0	effective overburden pressure of soils
686	P_{peak}	total penetration resistance of spudcan at peak
687	q_{net}	net bearing pressure
688	q_{peak}	net bearing pressure at peak in top layer soil
689	q_u	total bearing pressure
690	Q_v	penetration resistance of spudcan with an open cavity
691	$Q_{v,peak}$	penetration resistance of spudcan at peak with an open cavity

692	S_t	soil sensitivity
693	S_u	intact undrained shear strength of soil
694	S_{ub}	intact undrained shear strength of bottom layer soil
695	S_{ub0}	intact local undrained shear strength at spudcan base level in bottom layer
696	S_{ubs}	intact undrained shear strength of bottom layer soil at layer interface
697	S_{uc}	current undrained shear strength considering strain softening and rate effects
698	S_{ut}	intact undrained shear strength of top layer soil
699	t	thickness of top layer soil
700	T	thickness of top layer soil between spudcan base and initial layer interface
701	T'	equivalent thickness of soil plug (top layer soil) in bottom layer
702	V	volume of embedded spudcan foundation including shaft
703	V_p	intended full preload on spudcan foundation
704	z	depth below soil surface
705	ξ	accumulated absolute plastic shear strain
706	μ	rate parameter
707	γ'	effective unit weight of soil
708	$\dot{\gamma}$	maximum shear strain rate
709	$\dot{\gamma}_{ref}$	reference shear strain rate at which S_u is assessed
710	γ'_b	effective unit weight of bottom layer soil

711	γ'_t	effective unit weight of top layer soil
712	ξ_{95}	softening parameter
713	δ_{rem}	remoulded ratio (inverse of sensitivity)

714 **No. of Table: 2**

715 Table 1. Summary of centrifuge tests with corrected undrained shear strengths

716 Table 2. Summary of LDFE analyses performed for spudcan penetration

717

718

Table 1. Summary of centrifuge tests with corrected undrained shear strengths

Specimen	Test	D: m prototype	t: m	t/D	$\gamma'_t(\text{avg})$: kN/m ³	$\gamma'_b(\text{avg})$: kN/m ³	s_{ut} : kPa	s_{ub} : kPa	k: kPa /m
Event – 1 (E1UU)	I	T 1	6	0.25	7.2	7	14 (12.3) [#]	10 (11) [#]	0.26
		T 2	6	0.25					
		T 3	3	0.5			12 (10.2) [#]	8.5 (9.35) [#]	0.37
	II	T 4	6	0.75	7.35	7.15	21.3 (20.3) [#]	11 (13) [#]	0
		T 5	6	0.75					
		T 6	3	1.5			17 (16.6) [#]	11.2 (12.5) [#]	0
	III	T 7	6	1	7.5	7.3	20 (20) [#]	11 (12.3) [#]	0
		T 8	6	1					
		T 9	3	2			15 (15) [#]	11 (11.7) [#]	0
	IV	T 10	6	1.25	7.5	7.3	20 (20) [#]	12.2 (13.6) [#]	0
		T 11	6	1.25					
		T 12	3	2.5			17.5 (17.5) [#]	11.4 (12.7) [#]	0
Event – 2 (E2UU)	I	T 1	6	0.25	8.0	7.35	34 (26) [#]	11 (13.7) [#]	0
		T 2	6	0.25					
		T 3	3	0.5			23.8 (18.3) [#]	11.6 (13.7) [#]	0
	II	T 4	6	0.75	8.03	7.43	41 (38.3) [#]	9 (11) [#]	0
		T 5	6	0.75					
		T 6	3	1.5			25.5 (24.2) [#]	9.9 (12.4) [#]	0
	III	T 7	6	1	8.11	7.5	42.7 (42.7) [#]	12.1 (13.5) [#]	0
		T 8	6	1					
		T 9	3	2			26.6 (26.6) [#]	12.3 (13.7) [#]	0
	IV	T 10	6	1.25	8.13	7.75	47.3 (47.3) [#]	14 (14.7) [#]	0
		T 11	6	1.25					
		T 12	3	2.5			27.5 (27.5) [#]	15.3 (17) [#]	0

Specimen		Test	D: m prototype	t: m	t/D	$\gamma'_t(\text{avg}):$ kN/m ³	$\gamma'_b(\text{avg}):$ kN/m ³	S _{ut} : kPa	S _{ub} : kPa	k: kPa /m
Event – 1 (E1UNU)	I	T 1	6	1.5	0.25	7.5	7.25	16.2	3.6	1.55
		T 2	6		0.25			(12.5) [#]	(5.8) [#]	
		T 3	3		0.5			13 (10) [#]	3.8 (5.8) [#]	1.55
	II	T 4	6	4.5	0.75	7.5	7.25	21.4	7.5	2.6
		T 5	6		0.75			(20.5) [#]	(8.3) [#]	
		T 6	3		1.5			14 (13.4) [#]	7 (8) [#]	2
Event – 2 (E2UNU)	I	T 1	6	1.5	0.25	7.85	7.2	29.5	3	1.26
		T 2	6		0.25			(17.3) [#]	(3.5) [#]	
		T 3	3		0.5			24 (12.3) [#]	2.3 (2.5) [#]	1.34
	II	T 4	6	4.5	0.75	8.1	7.5	47	9.2	1.23
		T 5	6		0.75			(41) [#]	(11.5) [#]	
		T 6	3		1.5			31 (27) [#]	6.8 (7.5) [#]	1.54

719 [#] Original undrained shear strength measured using T-bar with a T-bar factor of 10.5

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Table 2. Summary of LDFE analyses performed for spudcan penetration

Analysis	$s_{\text{subs}}/s_{\text{ut}}$	t/D	kD/s_{subs}	$s_{\text{subs}}/\gamma'_b D$	S_t	Remarks
Group I	0.096	0.5	1.75	0.11	2.8	Comparison with centrifuge test data from Hossain and Randolph (2010a)
	0.2	0.75	0.8	0.2	2.8	
Group II	0.25~0.75	0.75	0.5	0.31	2.8	Parametric study
Group III	0.25 and 0.5	0.25~1.0	0	0.31	2.8	
Group IV	0.25	0.5 and 1.0	0~3.0	0.31	2.8	
Group V	0.5	1.0	0.5	0.31	1, 2.8 and 5	Effect of soil sensitivity S_t

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No. of Figure: 16

- Figure 1. Schematic diagram of embedded spudcan foundation in stiff-over-soft clay showing idealised open cavity and corresponding penetration resistance profile
- Figure 2. Numerical model used in parametric study
- Figure 3. Comparison between experimental and numerical results of T-bar penetration in stiff-over-soft clay
- Figure 4. Comparison between experimental and numerical results of spudcan penetration in stiff-over-soft clay (Group I, Table 2)
- Figure 5. Effect of strength ratio ($s_{\text{Subs}}/s_{\text{Sut}}$) on spudcan penetration resistance ($kD/s_{\text{Subs}} = 0.5$, $t/D = 0.75$, $s_{\text{Subs}}/\gamma' D = 0.31$, $\delta_{\text{rem}} = 1/S_t = 0.36$; Group II, Table 2)
- Figure 6. Effect of thickness ratio (t/D) on spudcan penetration resistance ($kD/s_{\text{Subs}} = 0$, $s_{\text{Subs}}/\gamma' D = 0.31$, $\delta_{\text{rem}} = 1/S_t = 0.36$; Group III, Table 2): (a) $s_{\text{Subs}}/s_{\text{Sut}} = 0.25$; (b) $s_{\text{Subs}}/s_{\text{Sut}} = 0.5$
- Figure 7. Effect of strength non-homogeneity (kD/s_{Subs}) of underlying layer on spudcan penetration resistance ($s_{\text{Subs}}/s_{\text{Sut}} = 0.25$, $s_{\text{Subs}}/\gamma' D = 0.31$, $\delta_{\text{rem}} = 1/S_t = 0.36$; Group IV, Table 2): (a) $t/D = 0.5$; (b) $t/D = 1.0$
- Figure 8. Effect of sensitivity on spudcan penetration resistance ($s_{\text{Subs}}/s_{\text{Sut}} = 0.5$, $t/D = 1.0$, $kD/s_{\text{Subs}} = 0.5$, $s_{\text{Subs}}/\gamma' D = 0.31$; Group V, Table 2)
- Figure 9. Design chart for normalised depth of peak resistance, d_p/D
- Figure 10. Relationship between predicted and measured or computed data of peak resistance using semi-empirical method: (a) Design chart for normalised peak resistance, $q_{\text{peak}}/s_{\text{Subs}}$; (b) Ratio between predicted and measured or computed q_{peak}
- Figure 11. Conceptual model for spudcan at punch-through in stiff-over-soft clay
- Figure 12. Relationship between predicted and measured or computed data of peak resistance using improved ISO method: (a) Design chart for thickness of equivalent soil plug in soft layer; (b) Ratio between predicted and measured or computed q_{peak}
- Figure 13. Design chart for normalised bearing capacity at layer interface, $q_{\text{int}}/s_{\text{Subs}}$
- Figure 14. Normalised bearing capacity profiles during spudcan penetration in bottom layer soft clay
- Figure 15. Design chart for deep bearing capacity factor N_{ed} in soft clay layer for spudcan penetration in stiff-over-soft clay
- Figure 16. Comparison between centrifuge test data and predicted penetration resistance profiles

Figure 1. Schematic diagram of embedded spudcan foundation in stiff-over-soft clay showing idealised open cavity and corresponding penetration resistance profile

Figure 2. Numerical model used in parametric study

Figure 3. Comparison between experimental and numerical results of T-bar penetration in stiff-over-soft clay

Figure 4. Comparison between experimental and numerical results of spudcan penetration in stiff-over-soft clay (Group I, Table 2)

Figure 5. Effect of strength ratio (s_{ubs}/s_{ut}) on spudcan penetration resistance ($kD/s_{ubs} = 0.5$, $t/D = 0.75$, $s_{ubs}/\gamma'D = 0.31$, $\delta_{rem} = 1/S_t = 0.36$; Group II, Table 2)

(a) $s_{ubs}/s_{ut} = 0.25$

(b) $s_{ubs}/s_{ut} = 0.5$

Figure 6. Effect of thickness ratio (t/D) on spudcan penetration resistance ($kD/s_{ubs} = 0$, $s_{ubs}/\gamma'D = 0.31$, $\delta_{rem} = 1/S_t = 0.36$; Group III, Table 2)

(a) $t/D = 0.5$

(b) $t/D = 1.0$

Figure 7. Effect of strength non-homogeneity (kD/s_{ubs}) of underlying layer on spudcan penetration resistance ($s_{\text{ubs}}/s_{\text{ut}} = 0.25$, $s_{\text{ubs}}/\gamma' D = 0.31$, $\delta_{\text{rem}} = 1/S_t = 0.36$; Group IV, Table 2)

Figure 8. Effect of sensitivity on spudcan penetration resistance ($s_{ubs}/s_{ut} = 0.5$, $t/D = 1.0$, $kD/s_{ubs} = 0.5$, $s_{ubs}/\gamma'D = 0.31$; Group V, Table 2)

Figure 9. Design chart for normalised depth of peak resistance, d_p/D

(a) Design chart for normalised peak resistance, $q_{\text{peak}}/s_{\text{ubs}}$

(b) Ratio between predicted and measured or computed q_{peak}

Figure 10. Relationship between predicted and measured or computed data of peak resistance using semi-empirical method

Figure 11. Conceptual model for spudcan at punch-through in stiff-over-soft clay

(a) Design chart for thickness of equivalent soil plug in soft layer

(b) Ratio between predicted and measured or computed q_{peak}

Figure 12. Relationship between predicted and measured or computed data of peak resistance using improved ISO method

Figure 13. Design chart for normalised bearing capacity at layer interface, q_{int}/s_{ubs}

Figure 14. Normalised bearing capacity profiles during spudcan penetration in bottom layer soft clay

Figure 15. Design chart for deep bearing capacity factor N_{cd} in soft clay layer for spudcan penetration in stiff-over-soft clay

Figure 16. Comparison between centrifuge test data and predicted penetration resistance profiles