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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 \( h_{p-T} \).
ISO Method

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

\[
Q_v = A \left\{ \frac{4T}{D} 0.75s_{ut} + \min \left( \frac{6 + 1.2 \frac{d_{int}}{D}}{9.0} s_{ub} + p'_{o} \right) \right\}
\]

\[
\leq A \min \left( \frac{6 + 1.2 \frac{d}{D}}{9.0} s_{ut} + p'_{o} \right)
\]

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

The ISO method was developed based on the results for a surface circular footing on uniform clays through model tests at 1g. The base of the soil plug is assumed to be fixed at the initial stiff-soft layer interface regardless of the spudcan penetration. As such, the soil plug carried down with the spudcan from the top (stiff) layer into the bottom (soft) layer, and corresponding contribution to the penetration resistance, are neglected. Additionally, the strain softening due to remoulding of clay is not explicitly considered, although a factor of 0.75 is applied on intact \(s_{ut}\) (see Equation 1) to obtain mobilised shear strength of the top 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 $\gamma'_t$, 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 $\gamma'_b$. $s_{ubs}$ 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 0.25\( D \)–1\( D \), 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, \( s_{sub}/s_{ut} \), was varied as 0.25–0.75. 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 \( s_{abs}/\gamma'_{b}D = 0.31 \) (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 \( kD/s_{abs} = 0-3 \).

**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 5\( D \) in horizontal and 8\( D \) 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 and 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

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

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 $\mu$ 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, $\xi$, from the intact condition to a fully remoulded ratio, $\delta_{\text{rem}} (= 1/S_t$, i.e. inverse of sensitivity $S_t$). The parameter, $\xi_{95}$, 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 $S_t = 2$–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 $\delta_{\text{rem}} s_{\text{sat}}$. 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, $q_{net}/s_{ub0}$, as a function of the normalised penetration depth, $(d - t)/D$, with $q_{net}$ calculated as

$$q_{net} = \frac{P - \gamma A}{\Lambda}$$

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 $s_{ubs}/s_{ut}$, 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 $s_{ubs}/s_{ut}$

To investigate the effect of the strength ratio $s_{ubs}/s_{ut}$ on the penetration resistance profile, Figure 5 is plotted for strength ratio $s_{ubs}/s_{ut} = 0.25, 0.3, 0.4, 0.5$ and $0.75$ with $t/D = 0.75$, $s_{ubs}/\gamma'D = 0.31$, and $kD/s_{ubs} = 0.5$ (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 $s_{ubs}/s_{ut}$ from 0.25 to 0.75.

The load-penetration response in the bottom layer is also affected by $s_{ubs}/s_{ut}$. 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 \( s_{ubs}/s_{ut} = 0.25 \) and 0.5, respectively, with \( s_{ubs}/\gamma' D = 0.31 \) and \( kD/s_{ubs} = 0 \) (Group III, Table 2). As \( t/D \) increases, the normalised peak resistance depth \( d_p/D \) becomes deeper, which is more profound for higher \( s_{subs}/s_{ut} \). For \( s_{ubs}/s_{ut} = 0.25 \), \( d_p/D \) increases from 0.12 to 0.17 (Figure 6a), whereas from 0.22 to 0.48 for \( s_{ubs}/s_{ut} = 0.5 \) (Figure 6b).

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

**Effect of Strength Non-homogeneity kD/s_{ubs}**

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

The effect of $kD/s_{\text{ubs}}$ on the bearing response in the bottom layer is significant particularly close to the layer interface. For high non-homogeneity factor of $kD/s_{\text{ubs}} = 3.0$, 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 $kD/s_{\text{ubs}} = 0$, 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 ($s_{\text{ubs}}/s_{\text{ut}} = 0.5$, $t/D = 1.0$, $kD/s_{\text{ubs}} = 0.5$ and $s_{\text{ubs}}/\gamma'D = 0.31$, 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 $q_{\text{peak}}/s_{\text{ubs}}$ 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 $\mu$ and $\xi_{95}$ may be obtained using Figure 8 of Hossain and Randolph (2009a). Through sensitivity analyses, Hossain and Randolph (2009a) quantified the influences of $\mu$ and $\xi_{95}$.

**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 ($d_p, q_{\text{peak}}$) in the top layer, the bearing capacity at the stiff-soft layer interface ($d_{\text{int}}, q_{\text{int}}$), and the deep bearing capacity factor $N_{\text{cd}}$. For the prediction of $q_{\text{peak}}$, 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, $s_{\text{ubs}}/s_{\text{ut}}, t/D$ and $kD/s_{\text{ubs}}$. The trend is shown in Figure 9 plotting the values from centrifuge tests and parametric study, which can be expressed as
Once \( \frac{d_p}{D} \) is determined, the corresponding penetration resistance at the peak can be calculated using either the semi-empirical method or the improved ISO method, as introduced below. Note, for layered clay deposits with uniform bottom (soft) layer \( (k = 0) \), Equation 4 indicates that \( \frac{d_p}{t} \) depends only on strength ratio \( \frac{s_{ubs}}{s_{uts}} \), i.e. independent of spudcan diameter, which is confirmed by Figure 6.

**Magnitude of peak resistance \( q_{peak} \)**

**Semi-empirical method**

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

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

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

**Improved ISO method**

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

\[
\frac{d_p}{D} = 1.3 \left( \frac{s_{ubs}}{s_{uts}} \right)^{1.5} \left( \frac{t}{D} \right)^{0.5} \left( 1 + \frac{kD}{s_{ubs}} \right)^{0.5} \leq \frac{t}{D}
\]
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, \( Q_{v,\text{peak}} \), 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 (4.2\( T's_{\text{ubs}}/D \); 3\( T's_{\text{ubs}}/D \) from the shear resistance around the plug and 1.2\( T's_{\text{ubs}}/D \) 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 (\( s_{\text{ub}} = s_{\text{ubs}} \)) can be expressed as

\[
[6] \quad Q_{v,\text{peak}} = A \left\{ 3 \frac{T}{D} s_{\text{st}} + \text{Min} \left[ 6 \left\{ 1 + 0.2 \frac{d_{\text{int}}}{D} \right\}, 9.0 \right] s_{\text{ub}} + p'_{0} + 4.2 \frac{T'}{D} s_{\text{ubs}} \right\}
\]

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 \( s_{\text{ub}} \) in Equation 6 as the average strength over the depth of \( D/2 \) below the layer interface. Calibration of the calculated values of \( Q_{v,\text{peak}} \) 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
It can be seen in Figure 12b that the proposed method predicts $q_{\text{peak}}$ 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 $k_D/s_{ubs}$ for $k_D/s_{ubs} \leq 1.5$. 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 $k_D/s_{ubs}$, 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 $k_D/s_{ubs}$ for $k_D/s_{ubs} > 1.5$. This is because higher $k_D/s_{ubs}$ 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 overestimation 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 $k_D/s_{ubs}$ 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 $k_D/s_{ubs} \leq 3$ 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).
Resistance at Layer Interface $q_{int}$

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

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

Deep Bearing Capacity Factor $N_{cd}$

For spudcan penetration resistance in the bottom layer, a simplified single layer approach is used, which implicitly incorporates the effect of the soil plug and corresponding additional resistance in the deep bearing capacity factor $N_{cd}$. To explore the value of $N_{cd}$, all profiles of $q_{net}/s_{ubs0}$ as a function of $(d - t)/D$ from numerical analyses and centrifuge tests are plotted in Figure 14 with a lower bound of 9.8 and an upper bound of 15.5. The corresponding stabilised values of $N_{cd}$ are plotted in Figure 15, with the best fit linear line expressed as

$$N_{cd} = 9.8 + 1.3 \left( \frac{s_{ubs}}{s_{ut}} \right)^{-1} \min \left( \left( \frac{t}{D} \right)^{0.5}, 1.0 \right) \left( 1 + \frac{kD}{s_{ubs}} \right)^{-1}$$

For the range of soil properties and layer geometries explored in this study, Equation 9 predicts the value of $N_{cd}$ mostly within an error of ±1.0. It should be noted that, in Equations 4, 5, 7, 8 and 9, exponents for $(s_{ubs}/s_{ut})$, $(t/D)$, and $(1 + kD/s_{ubs})$ are different according to their 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 \( q_{\text{peak}} \) using Equation 5 according to the semi-empirical method or \( Q_{v,\text{peak}} \) using Equation 6 combined with Equation 7 according to the improved ISO method. The total vertical reaction \( P_{\text{peak}} \) at \( d = d_p \) then can be calculated by adding the buoyancy \( \gamma'V \) to \( A_q_{\text{peak}} \), or by adding the buoyancy \( \gamma'V \) to \( Q_{v,\text{peak}} \) and then deducting the weight of the backfill soil above the spudcan. Once \( P_{\text{peak}} \) is determined, the potential for punch-through is assessed by comparing \( P_{\text{peak}} \) with the intended preload \( V_p \). If \( P_{\text{peak}} \leq V_p \), 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. \( V_p \leq 0.75P_{\text{peak}} \)). Otherwise, estimate the penetration resistances at the layer interface, \( (A_q + \gamma'V) \), and in the bottom layer, \( (A_{\text{cd},\text{sub}0} + \gamma'V) \), with \( q_{\text{int}} \) and \( N_{\text{cd}} \) calculated using Equations 8 and 9, respectively, and determine the punch-through distance \( h_{\text{PET}} \). The final spudcan-resting depth is determined by comparing \( (A_{\text{cd},\text{sub}0} + \gamma'V) \) 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 \( d = d_{\text{int}} \), 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 E2NU-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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LIST OF SYMBOLS

A  
spudcan plan area at largest section

D  
spudcan diameter at largest section

d  
penetration depth of spudcan base (lowest point at largest section)

d_{int}  
depth of stiff-soft layer interface

d_p  
depth of peak penetration resistance in top layer soil

E  
Young’s modulus

H_{cav}  
open cavity depth after spudcan installation

h_{p,T}  
punch-through distance

k  
rate of increase of shear strength within bottom layer

N_{cd}  
deep bearing capacity factor in the bottom layer soil

P  
total penetration resistance of spudcan

p'_0  
effective overburden pressure of soils

P_{peak}  
total penetration resistance of spudcan at peak

q_{net}  
net bearing pressure

q_{peak}  
net bearing pressure at peak in top layer soil

q_u  
total bearing pressure

Q_v  
penetration resistance of spudcan with an open cavity

Q_{v,peak}  
penetration resistance of spudcan at peak with an open cavity
S_t   soil sensitivity

s_u   intact undrained shear strength of soil

s_{ub}   intact undrained shear strength of bottom layer soil

s_{ub0}   intact local undrained shear strength at spudcan base level in bottom layer

s_{ubs}   intact undrained shear strength of bottom layer soil at layer interface

s_{uc}   current undrained shear strength considering strain softening and rate effects

s_{ut}   intact undrained shear strength of top layer soil

t   thickness of top layer soil

T   thickness of top layer soil between spudcan base and initial layer interface

T'   equivalent thickness of soil plug (top layer soil) in bottom layer

V   volume of embedded spudcan foundation including shaft

V_p   intended full preload on spudcan foundation

z   depth below soil surface

ξ   accumulated absolute plastic shear strain

μ   rate parameter

γ'   effective unit weight of soil

γ   maximum shear strain rate

γ_{ref}   reference shear strain rate at which s_u is assessed

γ'_{b}   effective unit weight of bottom layer soil
\( \gamma'_1 \) effective unit weight of top layer soil

\( \xi_{95} \) softening parameter

\( \delta_{\text{rem}} \) remoulded ratio (inverse of sensitivity)
### No. of Table: 2

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<td>Summary of LDFE analyses performed for spudcan penetration</td>
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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)

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) \( s_{ubs}/s_{ut} = 0.25; \) (b) \( s_{ubs}/s_{ut} = 0.5 \)

Figure 7. Effect of strength non-homogeneity (\( kD/s_{ubs} \)) of underlying layer on spudcan penetration resistance (\( s_{ubs}/s_{ut} = 0.25, s_{ubs}/\gamma'D = 0.31, \delta_{rem} = 1/S_{t} = 0.36; \) Group IV, Table 2): (a) \( t/D = 0.5; \) (b) \( t/D = 1.0 \)

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

Figure 9. Design chart for normalised depth of peak resistance, \( d_{p}/D \)

Figure 10. Relationship between predicted and measured or computed data of peak resistance using semi-empirical method: (a) Design chart for normalised peak resistance, \( q_{peak}/s_{ubs}; \) (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, \( q_{int}/s_{ubs} \)

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
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# Original undrained shear strength measured using T-bar with a T-bar factor of 10.5
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<th>$s_{ubs}/s_{ut}$</th>
<th>t/D</th>
<th>kD/$s_{ubs}$</th>
<th>$s_{ubs}/\gamma'_{b}D$</th>
<th>$S_t$</th>
<th>Remarks</th>
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Comparison with centrifuge test data from Hossain and Randolph (2010a)
Figure 1
Figure 2
Figure 3

Penetration depth of T-bar invert: m
Bearing pressure, $q_u$: kPa
Figure 4

- Bearing pressure, $q_u$: kPa
- Normalised penetration depth, $(d - t)/D$

Graph showing the relationship between bearing pressure and normalised penetration depth for different soil conditions and test types. The graph includes lines for Seabed, E2UNU-II-T 5, E2UNU-I-T 3, Stiff, Soft, Centrifuge test, and LDFE, corrected $s_u$.
Normalised bearing pressure, $q_{net}/s_{ub0}$

LDFE: $s_{ubs}/s_{ut} = 0.25, 0.3, 0.4, 0.5$ and $0.75$

(d – $t$)/$D = 0.2$

Figure 5
Figure 6

https://mc06.manuscriptcentral.com/cgj-pubs
Figure 7
Normalised bearing pressure, $q_{net}/s_{ub0}$

Normalised penetration depth, $(d - t)/D$

LDFE: $S_t = 5, 2.8$ and $1$

Seabed

Stiff

Soft

Figure 8
\[
\frac{d_p}{D} = 1.3 \left( \frac{s_{ubs}}{s_{ut}} \right)^{1.5} \left( \frac{t}{D} \right) \left( 1 + \frac{kD}{s_{ubs}} \right)^{0.5}
\]

Figure 9
Figure 10
Figure 11
Figure 12
\[
\frac{q_{int}}{s_{ubs}} = 10.2 \left( \frac{s_{ubs}}{s_{ut}} \right)^{-0.5} \left( \frac{t}{D} \right)^{0.25} \left( 1 + \frac{kD}{s_{ubs}} \right)^{0.25} \]

Figure 13
Figure 14

Normalized penetration depth, \( (d - t)/D \)

Normalized bearing pressure, \( q_{\text{net}}/s_{\text{ub0}} \)

Lower bound, \( N_{\text{cd}} = 9.8 \)

Upper bound, \( N_{\text{cd}} = 15.5 \)
Deep bearing capacity factor, $N_{cd}$

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

Figure 15
Figure 16