Bearing capacity on sand overlying clay: an analytical model for predicting post peak behaviour

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Abstract

New analytical models have recently been presented for evaluating the peak bearing capacity of circular footings penetrating sand-over-clay and the subsequent capacity once embedded in the underlying clay. However, in lieu of any analytical model, current methods of analysis use an inaccurate linear interpolation for the bearing capacity between the peak and the clay surface. This paper proposes an analytical model for calculating the bearing capacity of circular footings after the peak failure in sand-over-clay soils and before the footing enters the clay. Based on observations from visualisation experiments conducted in a geotechnical centrifuge, and consistent with the silo analysis approach used in predicting the peak capacity, the failure mechanism is a vertical frustum of sand progressively being forced into the underlying clay. Analysis of the capability of the new model to retrospectively simulate a database of 66 geotechnical centrifuge tests is provided. Motivation for improved prediction methods comes from the offshore mobile jack-up industry where concerns remain over the punch-through of large 20 m diameter spudcan footings in sand-over-clay. Accurate prediction of not just the peak bearing capacity but the subsequent load-displacement behaviour is required in order to assess the effectiveness of operational methods used to mitigate punch-through incidents.

Keywords

Bearing capacity, centrifuge modelling, spudcan, offshore engineering, sand, clay

Notation

d	penetration depth of footing
d_{peak}	footing depth at peak penetration resistance
$d_{\text{post, peak}}$	footing depth at local minimum of the post peak penetration resistance
D	diameter of the footing
D_F	distribution factor
E	parameter to simplify the algebra
H_s	sand thickness
I_D	relative density
N_c	bearing capacity factor
q	penetration resistance
q_0	surcharge at sand surface
q_{clay}	penetration resistance in the clay layer
q_{peak}	peak penetration resistance
$q_{\text{post, peak}}$	local minimum of the post peak penetration resistance
s_{um}	shear strength of clay at sand-clay interface
s_u	local undrained shear strength of clay
z	depth of the disc element
γ'_c	effective unit weight of clay
γ'_s	effective unit weight of sand in the disc element
ϕ'	friction angle of sand
ϕ^*	reduced friction angle due to non-associated flow rule
ϕ_{cv}	critical state friction angle of sand
ψ	dilation angle of sand
$\overline{\sigma}'_z$	mean vertical effective stress
τ	shear stress
σ'_n	normal stress

1 Introduction

Jack-ups are mobile self-elevating platforms consisting of a hull and three or more retractable legs. Each leg is supported by an independent circular ‘spudcan’ footing that can be over 20 m in diameter (with example shapes provided by Cassidy et al. [1]; Dean [2]). Once on location, the legs of a jack-up are jacked down into the seabed, and eventually the hull is raised above sea level, with the weight of the rig carried by the spudcans.

This installation procedure remains problematic for the offshore industry (Osborne and Paisley [3]; Hunt and Marsh [4]; Osborne et al. [5], [6]; Jack et al. [7]). In seabeds comprising sands overlying weaker clays a ‘punch-through’ event can cause a rapid vertical penetration of the spudcan. Uncontrolled punch-through of the sand into the clay, as illustrated in Fig. 1, poses a threat to the jack-up’s structure, stability and onboard personnel. Accurate pre-installation assessment of the vertical load-penetration curve remains an industry priority in predicting occurrence of punch-through events, with new analytical methods recently published (e.g. Okamura et al. [8]; Teh et al. [9]; Teh, [10]; Teh et al. [11]; Lee et al. [12]; Hu et al. [13]) aiming to improve the accuracy of industry guideline ISO 19905-1 (ISO) [14]. Cautiously installing the jack-up with the hull at positive draft or as close to the water as possible is an operational method currently used to mitigate the effects of a punch-through event. This allows any rapid leg penetration to cause hull immersion with the beneficial reduction of spudcan load due to buoyancy. However, to predict what will happen in such an event requires not just an accurate prediction method for the peak bearing capacity but all of the subsequent post-peak load-displacement behaviour.

This paper proposes an analytical model for calculating the bearing capacity of a spudcan after the peak failure in sand-over-clay soils and before the spudcan enters the clay. It is consistent with failure mechanisms observed through centrifuge tests (incl. Teh et al. [15], Hu et al. [16]). It is based within the analytical framework for calculating peak capacity by Lee et al. [12] and Hu et al. [13], and is calibrated against the database of 66 centrifuge tests reported by Teh et al. [17], Lee et al. [18] and Hu et al. [13],

[16]). All test results are provided in the supplementary material. This paper provides additional detail and accuracy to the method to calculate a full penetration resistance profile of Hu et al. ([19], [20]).

2 Predicting the penetration resistance profile

A typical spudcan penetration profile from a sand-over-clay centrifuge test is shown in Fig. 2. The penetration profile consists of a peak penetration resistance, q_{peak} , near to the sand surface, followed by an abrupt post-peak softening and then a gradual increase of penetration resistance as the spudcan approaches and penetrates the deeper clay. A local minimum of the post-peak penetration resistance, $q_{\text{post, peak}}$, was observed to occur well before the sand-clay interface.

In the full penetration profile prediction method of Hu et al. [20] (termed Hu et al. method hereafter), the spudcan penetration resistance profile is simplified as a linear interpolation between q_{peak} and the resistance in the clay layer, q_{clay} , as depicted in Fig. 2.

Based on observations from spudcan and flat footing penetration tests on dense sand overlying clay, a conceptual model was proposed by Lee et al. [12] to calculate q_{peak} . The model accounts for the stress level and dilatant response of sand. It was later modified by Hu et al. [13] to account for mobilisation embedment depth and validated experimentally for medium to dense sand and for spudcan geometry with base conical angle ranges from 0 to 21°. The analytical model assumes an inverted and truncated cone of sand, with an inclination angle equal to the dilation angle, is pushed down into the underlying clay (see Fig. 3, with evidence of this mechanism provided by Teh et al. [15]). Hence q_{peak} consists of the frictional resistance from the shearing sand, the weight of the sand frustum, and the bearing resistance from the underlying clay. The depth of peak resistance, d_{peak} , is taken as $0.12 H_s$, where H_s is the sand layer thickness.

When the spudcan penetrates to the underlying clay layer, the Hu et al. method expresses the bearing capacity factor N_c (ratio of q_{clay} to the local shear strength of the clay, s_u) as a linear function of H_s/D , where D is the diameter of the spudcan. Values of N_c were back-calculated from centrifuge tests and large deformation finite element (LDFE) analyses of the entire penetration process. The entrapped sand plug was

found to be cylindrical in shape with a height that averaged approximately $0.9H_s$ observed in both centrifuge testing and LDFE modelling (Hu et al. [21]).

Table 1 provides all the design equations of the Hu et al. method. However, no formulae for calculating the detailed load-displacement after the peak and before the sand-clay interface have been proposed by Hu et al., the ISO [14] or others. This is the focus of this paper.

3 Bearing capacity between $q_{\text{post, peak}}$ and the sand-clay interface

3.1 Depth of $q_{\text{post, peak}}$

Of the 66 sand-over-clay centrifuge tests summarised in the supplementary material, 40 tests displayed an obvious reduction in penetration resistance (and minimum $q_{\text{post, peak}}$) after q_{peak} . As shown in Fig. 4, $d_{\text{post, peak}}$ was consistently measured between 0.2 and 0.4 of the sand thickness H_s with an average of $0.31H_s$. Therefore, it is proposed that $d_{\text{post, peak}}$ be taken as $0.3H_s$ for simplicity. The other 26 centrifuge tests showed constant or increased resistance during the post-peak stage (see supplementary material), mainly observed in looser sand properties and comparatively stiffer clay underneath. Though these tests were not used in the derivation of $d_{\text{post, peak}}$, they were utilised in deriving the remainder of the conceptual model and are also considered in the retrospective analyses.

3.2 Failure mechanism between $q_{\text{post, peak}}$ and sand-clay interface

The normalised incremental deviatoric shear strain (the 2nd invariant of the tensor) contours at two penetration depths of $0.3H_s$ ($d_{\text{post, peak}}$) and $0.7H_s$ (intermediate depth between $d_{\text{post, peak}}$ and H_s) from a typical Particle Image Velocimetry (PIV) test are shown in Fig. 5 (test originally reported solely for peak capacity in Hu et al. [16], with $D = 8$ m, $H_s = 5.13$ m, relative density $I_D = 74\%$). Digital images were captured in centrifuge flight during penetration of a half-spudcan model against a transparent window of a strongbox. The images were then analysed using PIV techniques coupled with close-range photogrammetric correction of the measurements. This allowed interpretation of the failure mechanism after q_{peak} .

A surface of intensive shear strains forms around the periphery of the sand plug (Fig. 5a). With further penetration to $0.7H_s$ the plug extends beneath the sand-clay interface. Along each side of the spudcan and between the current depth of the spudcan and the original sand-clay interface, the maximum shear strain was approximately distributed along two vertical lines. This is consistent with the concept that with large amount of shear strain after q_{peak} , the sand has sheared beyond its peak strength into the critical state. The $q_{\text{post, peak}}$ is therefore consistent with the peak mechanism of Fig. 3, but with (i) the dilation angle, ψ , equal to zero (reducing the inclined shear slip sides to vertical) and the operative friction angle, ϕ' , equal to the critical state friction angle of sand, ϕ_{cv} , and (ii) a sand plug that penetrates into the clay. This is represented in the conceptual model of the post-peak penetration illustrated in Fig. 6.

3.3 Analytical model between $q_{\text{post-peak}}$ and sand-clay interface

The analytical treatment of the model follows the method of ‘differential slices’ with the sand frustum treated as a series of thin horizontal discs (following the approach for silo analysis, Walters [22], and in open ended piles, Randolph et al. [23]). Figure 6(b) shows the stress distribution on one of such infinitesimal disc elements. The element is acted on by mean vertical effective stress, $\bar{\sigma}'_z$ and $\bar{\sigma}'_z + \partial\bar{\sigma}'_z$ at the bottom, shear stress, τ , and normal stress, σ'_n , at the side. The effective unit weight of the disc element is γ'_s . The vertical force equilibrium on the element is:

$$\frac{\pi D^2}{4} \bar{\sigma}'_z + \frac{\pi D^2}{4} dz \gamma'_s = \frac{\pi D^2}{4} \left(\bar{\sigma}'_z + \frac{\partial \bar{\sigma}'_z}{\partial z} dz \right) + \pi D \tau dz \quad (1)$$

where z is the internal coordinate of the disc element within the sand frustum in the sand layer ($0 \leq z < 0.7H_s$). This equation can be simplified to the following differential equation:

$$\frac{\partial \bar{\sigma}'_z}{\partial z} + \frac{4\tau}{D} = \gamma'_s \quad (2)$$

For frictional material, the limiting shear stress along the slip surface can be written as:

$$\tau = \sigma'_n \tan \phi^* = \sigma'_n \frac{\sin \phi' \cos \psi}{1 - \sin \phi' \sin \psi} \quad (3)$$

where ϕ^* is the reduced friction angle, which is expressed in the form of ϕ' and ψ using the relationship given by Drescher and Detournay [24].

A distribution factor, D_F , which is defined as the ratio of normal stress to mean stress at the slip surface (same physical meaning as that in the q_{peak} formulations of Lee et al. [12] and Hu et al. [13]), is introduced. Thus, Equation 3 can be re-written as:

$$\tau = D_F \bar{\sigma}'_z \frac{\sin \phi' \cos \psi}{1 - \sin \phi' \sin \psi} \quad (4)$$

By substituting Equation 4 in Equation 2, for $\psi = 0$, $\phi' = \phi_{cv}$, Equation 2 can be expressed as:

$$\frac{\partial \bar{\sigma}'_z}{\partial z} + \frac{4D_F \sin \phi_{cv}}{D} \bar{\sigma}'_z - \gamma'_s = 0 \quad (5)$$

This can be integrated to give:

$$\bar{\sigma}'_z = \frac{\gamma'_s D}{4D_F \sin \phi_{cv}} + A \exp\left(-\frac{4D_F \sin \phi_{cv} z}{D}\right) \quad (6)$$

where A is a constant to be determined from the boundary condition.

Figure 7 presents three critical stages during spudcan penetration from $q_{\text{post, peak}}$ to the sand-clay interface. Referring to all stages in Fig. 7, when z is at its maximum $z_{\text{max}} (= H_s - d)$, the mean vertical effective stress $\bar{\sigma}'_z$ equals the bearing capacity of the clay, q_{clay} , with the general expression:

$$q_{\text{clay}} = N_c s_u + q_0 + \gamma'_s H_s + (\gamma'_c - \gamma'_s)(d - 0.1H_s) \quad (7)$$

where N_c is the bearing capacity factor; s_u is the local undrained shear strength of clay; q_0 is the surcharge at sand surface and γ'_c is the effective unit weight of the clay. The FE analysis in Lee [25] showed that the value of N_c for a shallow failure mechanism in the underlying clay appears little affected by the strength gradient of clay k . Therefore, following the approach simplified by Lee et al. [18] and Hu et al. [20], N_c has been back-calculated from the centrifuge tests with the best-fit expression being:

$$N_c = 14.8 \left(\frac{d - 0.1H_s}{D} \right) + 10.6 \quad \text{for } d \geq 0.3H_s \quad (8)$$

Replacing $\bar{\sigma}'_z$ with q_{clay} , A in Equation 6 can be evaluated as:

$$A = \left(q_{\text{clay}} - \frac{\gamma'_s D}{4D_F \sin \phi_{\text{cv}}} \right) \exp \left[\frac{4D_F \sin \phi_{\text{cv}} (H_s - d)}{D} \right] \quad (9)$$

Re-substituting A into Equation 6, the final equation for the mean vertical effective stress becomes:

$$\bar{\sigma}'_z = \frac{\gamma'_s D}{4D_F \sin \phi_{\text{cv}}} + \left\{ \left(q_{\text{clay}} - \frac{\gamma'_s D}{4D_F \sin \phi_{\text{cv}}} \right) \exp \left[\frac{4D_F \sin \phi_{\text{cv}} (H_s - d)}{D} \right] \right\} \exp \left(-\frac{4D_F \sin \phi_{\text{cv}} z}{D} \right) \quad (10)$$

The penetration resistance q as the spudcan penetrates from a depth of $0.3H_s$ to the sand-clay interface can be obtained by substituting Equation 7 and 8 into Equation 10 and setting $z = 0$, giving:

$$q = \frac{\gamma'_s D}{4D_F \sin \phi_{\text{cv}}} + \left\{ \left[\left(14.8 \frac{d - 0.1H_s}{D} + 10.6 \right) s_{\text{um}} + q_0 + \gamma'_s H_s + (\gamma'_c - \gamma'_s)(d - 0.1H_s) - \frac{\gamma'_s D}{4D_F \sin \phi_{\text{cv}}} \right] \exp \left[\frac{4D_F \sin \phi_{\text{cv}} (H_s - d)}{D} \right] \right\} \quad (11)$$

This equation can be further simplified as:

$$q = \frac{\gamma'_s D}{E} + \left\{ \left[\left(14.8 \frac{d - 0.1H_s}{D} + 10.6 \right) s_{\text{um}} + q_0 + \gamma'_s H_s + (\gamma'_c - \gamma'_s)(d - 0.1H_s) - \frac{\gamma'_s D}{E} \right] \exp \left[\frac{E (H_s - d)}{D} \right] \right\} \quad (12)$$

where

$$E = 4D_F \sin \phi_{\text{cv}} \quad (13)$$

Note that when the spudcan reaches the sand-clay interface ($d = H_s$), Equation 11 converges to Equation 7, confirming consistency of the solution.

3.4 D_F relationship for spudcan and flat footing

The parameter D_F has been determined from back-calculation of the centrifuge tests. When the 66 experimental $q_{\text{post, peak}}$ values were substituted into Equation 11, it was found that the back-calculated D_F

showed a power relationship with H_s/D , as shown in Fig. 8. It is assumed that D_F is constant with further penetration from $d_{\text{post, peak}}$.

For spudcan foundation:

$$D_F = 0.5 \left(\frac{H_s}{D} \right)^{-0.389} \quad \text{for } 0.3H_s \leq d < H_s \quad (14)$$

For a flat-based circular footing:

$$D_F = 0.4 \left(\frac{H_s}{D} \right)^{-0.063} \quad \text{for } 0.3H_s \leq d < H_s \quad (15)$$

By calculating the values at vertical increments from $d_{\text{post, peak}}$ to the clay surface ($0.1H_s$ increments used in this paper), the penetration resistance from $q_{\text{post, peak}}$ to the sand-clay interface is evaluated with Equations 12, 13 and 14 or 15.

4 Retrospective prediction of penetration resistance profiles

Retrospective predictions of the database of centrifuge experiments are used to assess the new analytical model. Four typical centrifuge tests, three dense sand tests (relative density, $I_D = 98\%$ and 92%) from Teh et al. [17] and Lee et al. [18] and one medium dense sand test ($I_D = 43\%$) from Hu et al. [13], are highlighted in Fig. 9. The equations used for the retrospective simulation are detailed in Table 1, which combines the previous Hu et al. method for q_{peak} and q_{clay} with the new analytical model of this paper.

The proposed post-peak method captures the trend of the profiles and it matches well with the experimental values, indicating a better overall prediction of the penetration resistance than the unrealistic linear interpolation. The curvature of the post-peak response is captured in the dense sand cases, as illustrated in Fig. 9a-c. The flatter, even slightly increasing capacity with depth, which is typical behaviour for medium dense sand over clay as shown in Fig. 9d, is also captured in the same model. If linear interpolation between the peak and sand-clay surface is assumed, as was in the previous Hu et al. method, a reducing load consistent with a potential punch-through failure is predicted in the Fig. 9d case. The new prediction method indicates less severity of failure and one that may be operationally mitigated in a controllable way.

The overall performance of the new analytical model in predicting post peak behaviour in the sand of all 66 sand overlying clay centrifuge tests is summarised in Fig. 10. For each testing series, the measured and predicted penetration resistances are presented with $q_{\text{calculated}}/q_{\text{measured}}$ against d/H_s ratio reported in 0.1 increments. The analytical model with the calibrated D_F provides an average of all the predictions of $q_{\text{calculated}}/q_{\text{measured}}$ of 0.98 and a standard deviation σ of 0.11. The majority of the predictions are within $\pm 20\%$ of the measured values. The linear regression line with a skew angle of $\theta = -0.13^\circ$, where θ is the angle between the linear regression line and the line of equality, indicates there is little skew for the predictions from $q_{\text{post, peak}}$ to the sand-clay interface. The performance indicators show that the analytical model and D_F expression describe the post-peak behaviour reasonably well.

Retrospective comparisons for the entire 66 centrifuge test database are provided in the supplementary material.

5 Conclusions

An updated analytical model for calculating the complete penetration profile of spudcan foundations on sand-over-clay has been presented. Specifically a new analytical model for post-peak behaviour of a penetrating spudcan in the sand layer is established based on simplified conceptual model and design equations. Straightforward algebraic expressions with clearly defined parameters are provided. Though presented here for spudcans the expressions are just as applicable to other circular footing applications penetrating sand-over-clay.

In retrospective simulations of the penetration resistance profiles of 66 centrifuge tests, it was shown that the calculated profiles match reasonably well with the experimental profiles. The improved formulation for the post-peak penetration resistance has been significantly enhanced and shown to match the variety of behaviours displayed in centrifuge tests of sands of different relative density. This new capability will be of use to the jack-up industry in combination with structural analysis methods to assess the effectiveness

of punch-through mitigation methods. For instance, a more accurate dynamic analysis of a rapidly penetrating spudcan, leg and hull system could be conducted (see Dean [2]).

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Table 1. Method for the bearing capacity-depth profile prediction

Section of load-penetration curve	Equations	Notes	References
Peak penetration	$q_{\text{peak}} = (N_{c0}s_{\text{um}} + q_0 + 0.12\gamma'_s H_s) \left(1 + \frac{1.76H_s}{D} \tan \psi \right)^{E^*} \quad (1)$ $+ \frac{\gamma'_s D}{2 \tan \psi (E^* + 1)} \left[1 - \left(1 - \frac{1.76H_s}{D} E^* \tan \psi \right) \left(1 + \frac{1.76H_s}{D} \tan \psi \right)^{E^*} \right]$ $N_{c0} = 6.34 + 0.56 \frac{k(D + 1.76H_s \tan \psi)}{s_{\text{um}}} \quad (2)$ $E^* = 2 \left[1 + D_F \left(\frac{\tan \phi^*}{\tan \psi} - 1 \right) \right] \quad (3)$ $D_F = 0.64 \left(\frac{H_s}{D} \right)^{-0.576} \quad \text{for } 0.16 \leq \frac{H_s}{D} \leq 1.00 \quad (4)$ $D_F = 0.62 \left(\frac{H_s}{D} \right)^{-0.174} \quad \text{for } 0.21 \leq \frac{H_s}{D} \leq 1.12 \quad (5)$ $\tan \phi^* = \frac{\sin \phi' \cos \psi}{1 - \sin \phi' \sin \psi} \quad (6)$	Equation (4) is for spudcan foundation and Equation (5) is for flat footing. The depth of peak resistance, d_{peak} , is $0.12 H_s$.	Lee et al. (2013b); Hu et al. (2014a)
From post-peak penetration to the clay surface	$q = \frac{\gamma'_s D}{E} + \left\{ \left[\left(14.8 \frac{d - 0.1H_s}{D} + 10.6 \right) s_u + q_0 + \gamma'_s H_s + \right] \right. \quad (7)$ $\left. \left(\gamma'_c - \gamma'_s \right) (d - 0.1H_s) - \frac{\gamma'_s D}{E} \right] \exp \left[\frac{E(H_s - d)}{D} \right] \right\}$ $E = 4D_F \sin \phi_{cv} \quad (8)$ $D_F = 0.50 \left(\frac{H_s}{D} \right)^{-0.389} \quad \text{for } 0.3H_s \leq d < H_s \quad (9)$ $D_F = 0.40 \left(\frac{H_s}{D} \right)^{-0.063} \quad \text{for } 0.3H_s \leq d < H_s \quad (10)$	Equation (9) is for spudcans and Equation (10) for flat footings. The depth of post-peak resistance is $0.3H_s$. The penetration depth d is in the range of $0.3H_s$ to H_s . Calculation can be repeated by increasing d at $0.1H_s$ increment.	This paper
Penetration in clay layer	$q_{\text{clay}} = N_c s_u + H_{\text{plug}} \gamma'_c = N_c s_u + 0.9H_s \gamma'_c \quad \left(0.16 \leq \frac{H_s}{D} \leq 1.00 \right) \quad (11)$ $N_c = 11 \frac{H_s}{D} + 10.5 \quad \left(0.16 \leq \frac{H_s}{D} \leq 1.12 \right) \quad (12)$	Calculation can be repeated for different penetration depth.	Hu et al. (2015b)

Fig. 1. Depiction of a punch-through event of a jack-up

Fig. 2. Illustrative spudcan penetration profile using centrifuge data and four critical penetration stages

Fig. 3. Modified failure-stress-dependent mechanism accounting for embedment depth attained during mobilisation of q_{peak} (modified after Hu et al. 2014a; point A in Fig. 2)

Fig. 4. Depth of penetration for $q_{\text{post, peak}}$

Fig. 5. Normalised incremental deviatoric shear strain contours at (a) depth of $0.3H_s$ and (b) depth of $0.7H_s$

Fig. 6. Schematic of spudcan penetration from $q_{\text{post, peak}}$ to the sand-clay interface and stress distribution on an infinitesimal disc element

Fig. 7. Schematic of three critical stages during spudcan penetration from $q_{\text{post, peak}}$ to the sand-clay interface

Fig. 8. Calibrated D_F from centrifuge tests (at $q_{\text{post, peak}}$ and $d = 0.3H_s$)

Fig. 9. Experimental and predicted bearing capacity-depth profiles

Fig. 10. Performance of the analytical model in predicting 66 sand overlying clay centrifuge tests