Numerical Modeling of Spudcan Deep Penetration in Three-layer Clays

J. Zheng¹, M. S. Hossain² and D. Wang³

¹ PhD Student, Centre for Offshore Foundation Systems (COFS), The University of Western Australia, 35 Stirling highway, Crawley, WA 6009, Tel: +61 (0)8 6488 4248, Fax: +61 (0)8 6488 1044, Email: 21102353@student.uwa.edu.au

² Corresponding Author, Associate Professor, ARC DECRA Fellow, Centre for Offshore Foundation Systems (COFS), The University of Western Australia, 35 Stirling highway, Crawley, WA 6009, Tel: +61 (0)8 6488 7358, Fax: +61 (0)8 6488 1044, Email: muhammad.hossain@uwa.edu.au

³ Assistant Professor, Centre for Offshore Foundation Systems (COFS), The University of Western Australia, 35 Stirling highway, Crawley, WA 6009, Tel: +61 (0)8 6488 3447, Fax: +61 (0)8 6488 1044, Email: dong.wang@uwa.edu.au

- Number of Words: 6830
- Number of Tables: 1
- Number of Figures: 14

Numerical Modeling of Spudcan Deep Penetration in Three-Layer Clays

Abstract: This paper reports the results from numerical modeling of spudcan deep penetration through three-layer clays. Two typical seabed strength profiles commonly encountered in the field, and identified as critical for potential punch-through failure, were considered. These include (a) uniform stiff-soft-stiff clay and (b) non-uniform clay with an interbedded stiff clay layer. 3D large deformation finite element (LDFE) analyses were carried out with and without simulating strain softening and strain rate dependency of the shear strength. The results were compared with previously published LDFE results and centrifuge test data. A detailed parametric study was undertaken, varying the relevant range of layer thicknesses (relative to the spudcan diameter), strength ratios, normalized strength and strength non-homogeneity. Punch-through and rapid leg penetration (for stiff-over-soft) and squeezing (for the reverse) were demonstrated by the penetration resistance profiles and associated soil failure mechanisms.

The results emphasized that the resistance profile from a spudcan continuous penetration in a three-layer sediment (e.g. stiff-soft-stiff) may be significantly different from the combined profiles resulted from two separate analyses on 1st-2nd layer and 2nd-3rd layer even if an appropriate depth factor was accounted for. For uniform stiff-soft-stiff clays, the presence of the bottom stiff clay layer within a short distance of 1 diameter affected spudcan bearing response in the top layer and suppressed the potential for punch-through failure (apart from rapid leg run for $t_1/D = 0.75$ and $s_{u1s}/s_{u2} = 3$). A trapped soil plug beneath the advancing spudcan, following penetration of the top stiff layer, caused the bottom stiff layer to be sensed earlier and led to enhance the limiting squeezing depth. For non-uniform clay with an

interbedded stiff layer, spudcan bearing response was dominated by squeezing (with limiting squeezing depth of about $\sim 0.18D$) in the top soft layer and potential punch-through and rapid leg run (for $t_2/D \geq 0.5$ and $s_{u2}/s_{u3s} = 2.5$) in the middle and bottom layers. In contrast to ISO recommendation, the soft soils in between the advancing spudcan base and the stronger layer did not squeeze out completely. Instead, some of the trapped material was forced into the underlying stronger layer. Punch-through distance was found to reduce significantly with increasing positive strength gradient of the bottom layer.

CE Database subject headings: Clays; Finite element method; Flow patterns; Foundations; Load bearing capacity; Offshore structures; Penetration resistance.

Introduction

Jack-up Rig and Spudcan Foundations

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; Bennett and Associates 2005; Randolph et al. 2005). Today's jack-ups typically consist of three independent truss legs, each attached to a large 10 to 20 m diameter spudcan. Prior to operations the legs are jacked down, eventually lifting the hull clear of the water and transferring the weight of the structure to the spudcan foundations on the seabed (Young et al. 1984). Each foundation is preloaded, by ballasting the hull, to between 50 and 100% above normal operating conditions to provide a margin of safety against failure during storm conditions (ISO 2011).

Jack-up Evolution in Problematic Seabed Sediments

Depletion of known reserves in the shallow waters of traditional hydrocarbon regions is resulting in exploration in deeper and undeveloped environments with more complex soil conditions. In emerging provinces and fields, highly layered soils are prevalent. For instance, over 75% of the case study data sets forming the basis for the InSafeJIP (2010) involved stratified seabed profiles, with interbedded layers of clay and sand displaying strong variations in shear strength. The Sunda Shelf, offshore Malaysia, Australia's Bass Strait and North-West Shelf, Gulf of Thailand, South China Sea, offshore India and Persian Gulf are particularly problematic in terms of stratigraphy and soil types (e.g. see Figure 1). Layered deposits are also encountered in the Gulf of Mexico (Menzies and Lopez 2011).

Identified Problems in Jack-up Installation

Highly stratified soil conditions with an interbedded strong layer overlying a weak layer (Figure 1) cause significant problems during jack-up installation and preloading of the spudcans. Depending on the applied preload relative to the peak bearing capacity (which is a function of the strength ratio and the relative thickness of the layers), sudden rapid penetration of the foundation may take place. This is termed either ‘rapid leg run’, which is controllable by jacking capacity, or ‘punch-through’ failure, which signifies uncontrollable penetration over a significant depth. Despite efforts by the jack-up industry to minimize these risks, punch-through failures continue to occur at an increasing rate (Jack et al. 2007, 2013). These events may lead to buckling of the leg, effectively decommissioning the platform, or may even result in toppling of the unit (Aust 1997; Maung and Ahmad 2000; Brennan et al. 2006; Kostelnik et al. 2007; Chan et al. 2008). In this study, spudcan penetration in two typical stratified seabed strength profiles commonly encountered in the field, and which are identified as critical for potential punch-through, are explored. The strength profiles include (i) uniform stiff-soft-stiff clay and (ii) non-uniform clay with an interbedded stiff layer, with typical examples shown in Figure 1.

Previous Work and Objective of Present Study

A number of investigations have been carried out on foundation response in two-layer clays through experimental work and finite element analyses, such as on stiff-over-soft clay (Brown and Meyerhof 1969; Wang and Carter 2002; Edwards and Potts 2004; Merifield and Nguyen 2006; Hossain and Randolph 2010a, b; Hossain et al. 2011; Tho et al. 2012) and in the reverse (Meyerhof and Chaplin 1953; Brown and Meyerhof 1969; Merifield and Nguyen 2006). Brown and Meyerhof (1969) conducted model tests at 1g using a circular footing sitting on the surface of layered soils. Small strain finite element (FE) analyses on surface footings were

undertaken by Edwards and Potts (2004) and Merifield and Nguyen (2006). Continuous penetration of plane strain and circular foundations were simulated by Wang and Carter (2002) using a large deformation finite element (LDFE) method, the RITSS (remeshing and interpolation technique with small strain; Hu and Randolph 1998) approach coupled with locally developed FE package AFENA (Carter and Balaam 1990).

Spudcan penetration on stiff-over-soft clays has been addressed more recently. The results from centrifuge model tests and large deformation finite element (LDFE) analyses (using RITSS/AFENA) were reported by Hossain and Randolph (2010a, b), while those from LDFE analyses, using the Coupled Eulerian-Lagrangian (CEL) approach in Abaqus/Explicit, were presented by Tho et al. (2012). They all presented the results in terms of load penetration response and/or of non-dimensional bearing capacity factors.

For spudcan penetration on stiff-over-soft clay, Hossain and Randolph (2010a, 2010b) have revealed the failure mechanisms. Severe punch-through was associated with vertically downward soil displacements beneath the spudcan in the upper layer and consequently a soil plug with the shape of an inverted truncated cone was eventually forced down into the underlying soft layer. Punch-through occurred for all cases where the strength ratio, s_{u1}/s_{u2} , was greater than or equal to 1.67, with the severity being greater the higher the strength ratio, and the thicker the upper layer, but reducing as the strength gradient in the lower layer increased. Design expressions have been developed by Brown and Meyerhof (1969) and Hossain and Randolph (2009c).

For spudcan penetration on soft-over-stiff clay, Merifield and Nguyen (2006) performed small strain finite element analyses for wished-in-place circular footings. The soft clay between the footing and the underlying strong layer tended to be squeezed out, but yielding also occurred well into the strong layer. From centrifuge model tests, Hossain et al. (2011) highlighted that a layer of soft clay from the 1st layer may be trapped at the base of the spudcan while

124 penetrating into the 2nd (stiff) layer. However, the design approach in ISO guidelines 19905-1
125 (ISO 2011) is based on bearing capacity factors proposed by Meyerhof and Chaplin (1953)
126 assuming that the cylindrical clay block between parallel rigid plates is squeezed out
127 completely.

128 To date, no research has been published that investigates penetration of spudcan foundations
129 in multi-layer clay profiles. As such, the recently finalized version of ISO guidelines 19905-1
130 (ISO 2011; Wong et al. 2012) and completed joint industry project ‘InSafeJIP’ (InSafeJIP
131 2010) provide no recommendation for assessing spudcan penetration in more general multi-
132 layer soils apart from noting that a so called ‘bottom-up approach’ can be used combining the
133 squeezing (for stiff over soft layering) and punch-through (for the reverse) criteria for two
134 layer systems.

135 In this study, an extensive investigation was carried out through centrifuge model tests and 3D
136 LDFE analyses on vertical penetration of spudcan foundations in general multi-layer fine grained
137 sediments. The results from centrifuge model tests were reported by Hossain et al. (2011) and
138 Hossain (2014). This paper provides results from parametric LDFE analyses considering a
139 simple elastic-perfectly plastic Tresca soil model, giving penetration resistance profiles and
140 soil failure mechanisms. Analyses were also performed using a modified Tresca model,
141 allowing strain softening, and incorporating strain rate dependency of the undrained shear
142 strength, for comparison. The difference between combining two-layer soft-over-stiff and
143 stiff-over-soft systems (bottom-up approach) and a spudcan continuous penetration in a three-
144 layer soil is highlighted.

Numerical Analysis

Geometry and Parameters

This study has considered a circular spudcan of diameter D , penetrating into a three-layer clay deposit as illustrated schematically in Figure 2, where the middle layer with uniform undrained shear strength s_{u2} and thickness t_2 is sandwiched by the top layer of non-uniform undrained shear strength $s_{u1} = s_{um} + k_1z$ and thickness t_1 ; and the bottom layer of non-uniform undrained shear strength $s_{u3} = s_{u3s} + k_3(z - t_1 - t_2)$, with undrained shear strength at the 2nd-3rd layer interface s_{u3s} , and (nominally) infinite depth. s_{u1s} is the undrained shear strength at the 1st-2nd layer interface (i.e. $s_{u1s} = s_{um} + k_1t_1$). 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 (see Figure 2). 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).

The numerical program concentrated on simulating simplified strength profiles from reported case histories (e.g. InSafeJIP 2010) with normalized soil properties and layer geometries. In addition, in order to select realistic soil parameters for parametric study, a survey was carried out through offshore geotechnical characterization reports. The three-layer geometries considered here are commonly encountered in the Gulf of Thailand and Sunda Shelf, including Java Sea, as reported by Castleberry II and Prebaharan (1985), Handidjaja et al. (2004), Kostelnik et al. (2007), Chan et al. (2008) and InSafeJIP (2010).

For uniform three-layer clay sediments, the undrained shear strength of stiff clay ranges from 40 to 120 kPa, while that of soft clay varies between 10 and 40 kPa. For non-uniform clay deposit with an interbedded uniform clay layer, typical undrained shear strength of the middle layer can be found as $s_{u2} = 40$ to 75 kPa, and those of the 1st and 3rd layers as $s_{um} = 0$ to 15 kPa and $s_{u3s} = 30$ to 50 kPa, respectively, increasing with depth with a gradient $k_1 = k_3 = 1\sim3$

kPa/m. The effective unit weight γ' for a three-layer soil system has a typical range of 6.5 to 8.5 kN/m³. The (equivalent area) diameter of modern spudcan foundations ranges from 10 to 20 m (Menzies and Roper 2008). The selected parameters for this study are assembled in Table 1, encompassing a range of practical interest in terms of the likelihood of rapid leg run or punch-through.

Analysis Details

The LDFE analyses were performed using the CEL approach in Abaqus/Explicit. The feature of ‘fixed’ Eulerian mesh and the contact between the Eulerian and Lagrangian materials allow for effective application of the CEL approach in large deformation geotechnical problems, circumventing some corresponding problems such as severe distortion of mesh associated with the progress of an object penetration. 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).

Current CEL function is only for three-dimensional elements. For the parametric studies, only a quarter sector of the domain was involved in the analyses accounting for the symmetry inherent, as shown in Figure 3. The soil domain was chosen as 6.25D in horizontal and 11.7D in depth, which was testified as sufficiently large to avoid boundary effect. Similar domain sizes were considered by Qiu and Henke (2011) and Tho et al. (2012) in spudcan continuous penetration analysis. The spudcan was modeled as a rigid Lagrangian body and the soil-spudcan interface as fully rough unless otherwise stated. The soil and spudcan foundation were discretized using Eulerian hexahedral elements of type EC3D8R and continuum elements of types C3D8R (hexahedron) and C3D6 (triangular prism), respectively. These element types are linear elements with reduced integration and hourglass control. The soil

layers were included in the Eulerian mesh, and they underlay an initially ‘void’ layer to accommodate the soil heave resulting from spudcan penetration.

The soil was modeled as a linear elastic-perfectly plastic material obeying a Tresca yield criterion. This simple soil model provided satisfactory results in terms of ultimate penetration resistance in comparison to theoretical solutions for various problems such as spudcan (Hossain and Randolph 2009b), circular and strip footings (Wang and Carter 2002) and ball and cone (Lu et al. 2004; Zhou et al. 2012) penetration in clay and uplift of plate anchors (Thorne et al. 2004). The elastic parameters for clay were considered to be independent of stresses and a uniform stiffness ratio of $E/s_u = 200$ was taken throughout the clay profile. The stiffness ratio is within the range commonly adopted for soft clays, but the precise value has negligible effect on the results presented. Considering relatively fast penetration of spudcans in the field, all the analyses simulated undrained conditions and adopted a Poisson’s ratio $\nu = 0.49$ (sufficiently high to give minimal volumetric strains, while maintaining numerical stability). The geostatic stress conditions were modeled using $K_0 = 1$, as the stable penetration resistance (once backflow is fully established) has been found to be unaffected by the value of K_0 (Zhou and Randolph 2009).

Typical computation times on a fast desktop computer were about 36~48 h for a spudcan penetration of three diameters. This is slightly longer than the time required for undertaking an axisymmetric analysis using e.g. RITSS/AFENA. However, 3D LDFE analyses will provide with the long term opportunity for carrying out further analyses under operational loadings and for spudcan penetration near an existing footprint.

Incorporation of Combined Effect of Strain Rate and Softening

Following Einav and Randolph (2005) and Zhou and Randolph (2007), the Tresca soil model was extended to take the combined effects of rate dependency and progressive softening into

account. The undrained shear strength at each integration point was modified at the beginning of each time step, based on the average rate of shear strain over the last time step, $\dot{\gamma}$, and the current accumulated absolute plastic shear strain, ξ , according to

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

where s_{uc} is the 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 shear strain rate relative to a reference value, $\dot{\gamma}_{ref}$, which was considered as 1.5%/h corresponding to the typical strain rate in triaxial tests (Erbrich 2005; Lunne et al. 2006). The augment of shear strength follows a logarithmic law with rate parameter μ taken as 0.1 for spudcan foundations (Low et al. 2008). The spudcan penetration rate in the field was taken as 2 m/hour.

The second part of Equation 1 models the degradation of strength according to an exponential function of cumulative shear strain, ξ , from the intact condition to a fully remolded ratio, δ_{rem} ($= 1/S_t$, i.e. inverse of sensitivity S_t). The relative ductility is controlled by the parameter, ξ_{95} , which represents the cumulative shear strain required for 95% remolding. A typical value of $\xi_{95} = 12$ (i.e., 1200% shear strain; Randolph 2004) was used. Further details can be found in Hossain and Randolph (2009a) and Zheng et al. (2013).

Validation of Results

The load-penetration responses are presented in terms of net bearing pressure, q_{net} , normalized by the shear strength intercept at the 2nd-3rd layer interface (s_{u3s}), as a function of normalized penetration depth, d/D , with q_{net} calculated by deducting spudcan buoyancy as

$$q_{\text{net}} = \frac{P}{A} - \gamma' \frac{V}{A} \quad (2)$$

where P is the penetration resistance (i.e. the total vertical reaction force provided by the soil), V is the volume of the embedded spudcan including shaft and A is the largest plan area of the spudcan. Calculation of q_{net} using Equation 2 and normalization using s_{u3s} yield a deep (after backflow) bearing capacity factor in the 3rd layer of uniform undrained shear strength and allowed to maintain the form of the resistance profile for all soil profiles considered. In all the response curves, the spudcan penetration is plotted relative to the depth of the lowest maximum diameter of the spudcan below the original ground surface (i.e. $d/D = 0$, see Figure 2).

Validation against Results from LDFE Analyses Using RITSS

Validation exercise was carried out against existing results from LDFE analyses and centrifuge tests. As noted previously, Wang and Carter (2002) and Hossain and Randolph (2010b) undertook axisymmetric LDFE analyses using the RITSS coupled with AFENA. To compare with their work, analyses were performed for uniform two-layer ($k_1 = k_2 = 0$) stiff-over-soft clays, with and without soil weight, and without accounting for the effects of strain softening and rate dependency.

Figure 4a shows the normalized net bearing pressure, $q_{\text{net}0}/s_{u2}$ (where the subscript 0 denotes weightless soil) for a smooth-based circular flat footing penetrating into layered soils with $t_1/D = 1$ and $s_{u1s}/s_{u2} = 1, 2$ and 10 (Group I, Table 1). The figure also includes results reported

by Wang and Carter (2002). Both sets of results are in excellent agreement with variations mostly less than 5%, confirming the accuracy of the CEL results.

For spudcan penetration, LDFE analyses were performed on weighty stiff-over-soft clay with $t_1/D = 0.5$ and $s_{u1s}/s_{u2} = 1.25, 1.67, 2.5$ and 5 (Group II, Table 1). The normalized penetration resistances are plotted in Figure 4b for comparison with the results reported by Hossain and Randolph (2010b). Again excellent agreement can be seen in terms of both soil backflow depth and load-penetration response with variations in both regards mostly less than 5%.

Validation against Centrifuge Test Data

Comparisons are presented here between two of the model tests (Tests T6 and T8, Table 2 of Hossain et al. 2011) and LDFE simulations (Group III, Table 1). Tests modeled a spudcan of prototype diameter 12 m penetrating three-layer deposits with $t_1/D = 0.38$, $t_2/D = 0.58$ (for T6) and $t_1/D = 0.42$, $t_2/D = 0.5$ (for T8). The undrained shear strength profiles were idealized as $s_{u1} = s_{u3} = 2 + 0.6z$ kPa, $s_{u2} = 23$ kPa (for T6) and $s_{u1} = 16 + 0.8z$ kPa, $s_{u2} = 11$ kPa, $s_{u3} = 33$ kPa (for T8). The average clay effective unit weights were 7.5 and 7.3 kN/m³ for the top layer and lower layers respectively. Analyses were performed for the above cases for idealized Tresca materials (without considering softening and rate-dependency), and also with strain softening and rate dependency taken into account ($\delta_{rem} = 0.36$, $\mu = 0$ and 0.1, $\xi_{95} = 12$, $\dot{\gamma}_{ref} = 1.5\%/h$).

Figures 5a and 5b show the load-penetration responses from the LDFE analyses and centrifuge tests. For rate-independent, perfectly plastic (referred to as ‘ideal’) soil, the LDFE result is in good agreement with the centrifuge test data at shallow penetration depths in the top layer and lies above (around 15~20% higher) at deep penetration depths in the 2nd and 3rd layers. Taking strain softening and strain rate effects into account, the curves diverge gradually with the progress of penetration, and eventually closely match the one from the

centrifuge test. Furthermore, the form of the penetration resistance profile and the likelihood of punch-through are better reflected in the curves for strain softened, rate dependent soils.

Results and Discussion

The soil failure mechanisms and hence the bearing capacity profiles of spudcan foundations on layered clays are affected mainly by the following factors: (a) the strength ratios between adjacent layers, s_{u1s}/s_{u2} and s_{u2}/s_{u3s} ; (b) the thickness ratios of soil layers relative to the spudcan diameter, t_1/D and t_2/D ; (c) the normalized undrained shear strength of the bottom layer, $s_{u3s}/\gamma'D$; (d) the strength gradient of the bottom layer, denoted by the non-homogeneity factor, k_3D/s_{u3s} . In the following sub-sections, critical soil failure mechanisms for the two soil profiles investigated are first presented. The penetration resistance profiles for ideal soil are then discussed in relation to the various factors, and finally the influence of accounting for softening and rate dependency is highlighted. For selecting computational aspects, routine preliminary simulations were carried out, varying the element size around the spudcan, h_{min}/D , and penetration velocity of the spudcan, v_{sp} . Balancing the computational effort and accuracy of the computed results, and also complying with existing studies (e.g. Tho et al. 2012), $h_{min}/D = 0.05$ and $v_{sp} = 0.1$ m/s were adopted for the parametric study.

Soil Failure Mechanisms

As noted previously, a spudcan penetration in three-layer stiff-soft-stiff clay deposits is usually treated combining spudcan penetration in two-layer stiff-over-soft and the reverse deposits. This approach essentially neglects the influence of the overlying layer, which is highlighted below.

The soil flow mechanisms observed are discussed below, linking these directly to the profile of penetration resistance. Figure 6a depicts the soil flow mechanism at a spudcan penetration depth of $d/D = 0.48$ (near the 1st-2nd layer interface) in a deposit with $s_{u1} = s_{u1s} = 120$ kPa, $s_{u2} = 40$ kPa, $s_{u3} = s_{u3s} = 75$ kPa and $t_1/D = t_2/D = 0.5$ (Group IV, Table 1). A somewhat rapid leg run occurs in the upper layer (see the corresponding resistance profile later in Figure 7b). A soil plug from the upper stiff layer is trapped beneath the advancing spudcan and forced down into the 2nd layer (see Figure 6a). The advancement of the spudcan with the trapped soil plug at the base results in early squeezing of soil sensing the 3rd layer, as shown in Figure 6a. The resistance profile therefore rises in the middle soft layer without tending to reduce.

For spudcan penetration in soft-over-stiff clay, it is assumed in the ISO guideline (ISO 2011) that the soil between the advancing spudcan and the stiff clay layer squeezes out completely. Figure 6b shows the soil flow mechanism at a spudcan penetration depth of $d/D = 0.55$ (after penetrating into the 3rd layer) in a deposit with $s_{u1} = 2 + 2z$ kPa, $s_{u2} = 75$ kPa, $s_{u3} = 30 + 1(z - t_1 - t_2)$ kPa and $t_1/D = t_2/D = 0.25$ (Group V, Table 1). Clearly, a layer of soft clay from the 1st layer is trapped beneath the spudcan base and penetrated into the 3rd soft layer, enhancing the height of the soil plug and hence the corresponding frictional resistance.

Penetration Resistance: Uniform Clay with an Interbedded Soft Clay Layer

Effect of strength ratio (s_{u1s}/s_{u2})

In an attempt to explore the effect of strength ratio s_{u1s}/s_{u2} on the form of the penetration resistance profile, the results of various top layer strength $s_{u1} = s_{u1s} = 50, 80$ and 120 kPa are plotted in Figures 7a~7c for thickness ratio $t_1/D = 0.75, 0.5$ and 0.25 respectively. The strengths of the 2nd and 3rd layers are constant as $s_{u2} = 40$ kPa and $s_{u3} = s_{u3s} = 75$ kPa ($s_{u2}/s_{u3s} = 0.53$) with the normalized strength of the bottom layer $s_{u3s}/\gamma'D = 0.78$ (Group IV, Table 1).

For given thickness ratios of t_1/D and t_2/D , the bearing pressure increases as the strength of the top layer and (hence) the strength ratio s_{u1s}/s_{u2} increase. This is because the proportion of domination of the top strong layer increases and that of the middle soft layer decreases. This influence becomes more profound with increasing thickness of the top layer, t_1/D , and/or reducing that of the 2nd layer, t_2/D .

The critical depth of soil backflow above the advancing spudcan (which can also be taken as the limiting cavity depth) is also affected by the strength ratio although, in all the analyses performed in Group IV (Table 1), backflow occurs in the middle or bottom layer. A stronger top layer strength has delayed soil backflow, with more downward soil deformation and forming thicker soil plug at the base of the advancing spudcan.

For all cases, no reduction in penetration resistance profile forming a local peak is evident. The results for identical strength ratio, s_{u1s}/s_{u2} , and thickness ratio, t_1/D , but in the absence of the 3rd layer (i.e. on two-layer stiff-over-soft clay deposits) are also included in Figures 7a and 7b for comparison. In contrast to the results for three-layer sediments, a reduction in bearing resistance profile occurs for stiff-over-soft clay deposits, which is consistent with Hossain and Randolph (2010a, 2010b). This indicates that, for the cases investigated in this study, spudcans sense the influence of the bottom strong layer while penetrating in the top layer. This influence becomes more profound as the thickness of the middle layer reduces. In terms of soil backflow, the presence of the third stiff layer squeezes the soil underneath the spudcan and thus reduces the depth of soil backflow.

In addition, the penetration resistance profiles from spudcan penetration on two-layer soft-over-stiff clay (i.e. s_{u2} overlying s_{u3} with the absence of the 1st layer; $s_{u2} = 40$ kPa, $s_{u3s} = 75$ kPa, $t_2/D = 0.25$) are also included in Figure 7a starting from the 1st-2nd layer interface, with and without adding a depth factor of 0.9 (i.e. $1 + 0.2t_1/D$ following ISO 2011) to the original

normalized net bearing pressure. It can be seen that the penetration resistance profile combining the results on two-layer stiff-over-soft and soft-over-stiff clay deposits produces a different form, with significantly lower bearing resistance, compared to the one from continuous penetration analysis on three-layer stiff-soft-stiff clay deposit.

Effect of thickness ratios (t_1/D and t_2/D)

In order to explore the effect of thickness ratios on the form of load-penetration response, the profiles of $t_2/t_1 = 0.33, 1.0$ and 3.0 are plotted in Figures 8a~8c, but keeping $(t_1 + t_2)/D = 1.0$, with various strengths of the top layer of $s_{u1} = s_{u1s} = 120, 80$ and 50 kPa respectively (Group IV, Table 1). Again, the strengths of the 2nd and 3rd layers were constant as $s_{u2} = 40$ kPa and $s_{u3} = s_{u3s} = 75$ kPa ($s_{u2}/s_{u3s} = 0.53$).

The effect of thickness ratios becomes more profound as s_{u1s}/s_{u2} increases or the difference of the undrained shear strengths between 1st and 2nd layer increases. For instance, Figure 8a shows the penetration resistance profiles and soil flow mechanisms for $s_{u1s}/s_{u2} = 3$ ($s_{u1s} = 120$ kPa, $s_{u2} = 40$ kPa, $s_{u3s} = 75$ kPa). The depth of soil backflow and the bearing pressure increase with increasing t_1/D . When the spudcan was at the 2nd-3rd layer interface, a soil plug was forced down into the bottom layer (see inserts of Figure 8a). The trapped plug comprised materials from both 1st stiff layer and 2nd soft layer. With increasing t_1/D , more soil from the 1st stiff layer was trapped underneath the spudcan, while the quantity of the trapped soil from the 2nd soft layer was nearly consistent regardless of the thickness of that layer (t_2/D), forming thicker soil plug. Furthermore, as all the curves tend to approach the steady ultimate resistance corresponding to the bottom layer, with increasing thickness ratio (t_1/D), the resistance profile becomes stiffer showing greater tendency of occurring rapid leg run.

For $s_{u1s}/s_{u2} = 1.25$ ($s_{u1s} = 50$ kPa, $s_{u2} = 40$ kPa, $s_{u3s} = 75$ kPa; Figure 8c), all curves somewhat merge together with negligible effect of thickness ratios. This indicates that, as the difference

between the strengths of the 1st layer and 2nd layer is small, the three-layer stiff-soft-stiff clay profile has turned to two-layer soft-over-stiff clay profile. As such, squeezing mechanism dominates the behavior with the proximity to the bottom layer.

For the cases just discussed, the total thickness of the first two layers were kept constant as $(t_1 + t_2)/D = 1.0$. An analysis was also carried out for $t_1/D = t_2/D = 0.75$ (i.e. $(t_1 + t_2)/D = 1.5$) with strength ratio $s_{u1s}/s_{u2} = 3$ ($s_{u1s} = 120$ kPa, $s_{u2} = 40$ kPa, $s_{u3s} = 75$ kPa). Figure 9 shows the result along with the curves for $t_1/D = 0.75$, $t_2/D = 0.25$ and $t_1/D = 0.25$, $t_2/D = 0.75$ for comparison. For identical t_2/D , a thicker 1st stiff layer allowed for developing the capacity of that layer and, for identical t_1/D , a thicker middle soft layer allowed for reducing the resistance back to the capacity of the middle layer as well as attracting the soil deformation avoiding the influence of the bottom strong layer (up to some extent). The resultant is that the profile for $t_1/D = t_2/D = 0.75$ is bracketed by the curves for $t_1/D = 0.75$, $t_2/D = 0.25$ and $t_1/D = 0.25$, $t_2/D = 0.75$ in the 1st layer and shows a potential for punch-through at $d/D = 0.25$.

The results from numerical analysis of a rough spudcan penetration in uniform clays with undrained shear strength of s_{u2} and s_{u3} together with the plasticity solutions (Martin and Randolph 2001) for a buried plate (both smooth and rough) in clay are also included in Figures 7~9 for comparison. It is apparent that the thickness of the middle layer of up to $t_2/D = 0.75$ was not sufficient for reducing the response back to the profile for uniform clay with strength = s_{u2} , although they tend to do so for lower s_{u1s}/s_{u2} and higher t_2/D . In the bottom layer, all curves converge and tend to merge together with the profile for uniform clay with strength = s_{u3} . In some cases slightly higher resistance was resulted due to the additional frictional resistance mobilized around the periphery of the trapped soil plug. The limiting normalized bearing resistance in the bottom layer (without the effect of the soil plug) is ~3% lower than the exact plasticity solution for a rough-based buried plate (Martin and Randolph 2001).

Effect of normalized strengths ($s_{u2}/\gamma'D$ and $s_{u3s}/\gamma'D$)

In order to explore the effect of normalized strength on spudcan bearing behavior, two curves for $s_{u2}/\gamma'D = 0.083$, $s_{u3s}/\gamma'D = 0.16$ ($s_{u1s} = 24$ kPa, $s_{u2} = 8$ kPa, $s_{u3s} = 15$ kPa) and $s_{u2}/\gamma'D = 0.42$, $s_{u3s}/\gamma'D = 0.78$ ($s_{u1s} = 120$ kPa, $s_{u2} = 40$ kPa, $s_{u3s} = 75$ kPa), with all normalized parameters consistent ($s_{u1s}/s_{u2} = 3$, $s_{u2}/s_{u3s} = 0.53$, $t_1/D = t_2/D = 0.75$; see Group IV, Table 1), are displayed in Figure 10. The form of these two resistance profiles is completely different. For $s_{u2}/\gamma'D = 0.42$, $s_{u3s}/\gamma'D = 0.78$, as discussed previously, the curve shows a potential for rapid leg run in the top layer, followed by rises monotonically with depth. By contrast, for $s_{u2}/\gamma'D = 0.083$, $s_{u3s}/\gamma'D = 0.16$, i.e. for significantly lower normalized undrained shear strengths, the penetration resistance profile rises monotonically with depth in the 1st stiff layer. It drops sharply forming a local peak in the middle soft layer with the initiation of soil backflow. This may lead to a punch-through failure in the field during installation of jack-ups in similar sediments. This is consistent with the finding reported by Hossain and Randolph (2010b) although for stiff-over-soft clays.

Effect of accounting for strain rate and softening

A comparison in terms of the form of the penetration resistance profile with ($\delta_{rem} = 0.3$, $\mu = 0.1$, $\xi_{95} = 12$, $\dot{\gamma}_{ref} = 1.5\%/h$) and without accounting for strain rate and softening effects are shown in Figure 11 ($s_{u1s} = 50$ and 120 kPa, $s_{u2} = 40$ kPa, $s_{u3s} = 75$ kPa). It may be seen that the profiles for strain softened, rate dependent soil exhibit potential for rapid leg run in the 2nd layer. As discussed previously, negligible or no rapid leg run occurs for ideal soil. In addition, bearing resistance is around 17% lower particularly at deep penetration depths.

Penetration Resistance: Non-uniform Clay with an Interbedded Stiff Clay Layer

In the analyses on non-uniform clay with an interbedded stiff clay layer, the strength of the stiff (2nd) layer (for Group V, Table 1) and the degree of non-homogeneity of the 3rd layer (for Group VI, Table 1) were varied.

Effect of strength ratio (s_{u2}/s_{u3s})

To demonstrate the effect of the 2nd layer strength or strength ratio s_{u2}/s_{u3s} on the form of the penetration resistance profile, Figures 12a~12c are plotted for $s_{u2}/s_{u3s} = 1.33$ and 2.5, with the thickness ratios $t_1/D = 0.25$ and $t_2/D = 0.25, 0.5$ and 0.75, respectively. The strength non-homogeneity factor for the top and bottom layers are $k_1D/s_{u1} = 12$ and $k_3D/s_{u3s} = 0.4$ ($s_{u1} = 2 + 2z$ kPa, $s_{u3} = 30 + 1(z - t_1 - t_2)$ kPa; see Group V, Table 1). In the top layer, squeezing dominates the response due to the effect of significantly stiffer (strength ratio of $s_{u1s}/s_{u2} = 0.11\sim 0.2$; as also described by Hossain et al. (2011) and Hossain 2014) and relatively thicker (thickness ratio of $t_2/D = 0.25\sim 0.75$) 2nd layer.

It is of interest to identify the depth range above the strong layer where squeezing effects start to increase the penetration resistance (defined as limiting squeezing depth). This limiting squeezing depth is a function of various factors such as the strength ratio between the two layers and whether a plug is being carried down with the advancing spudcan (as was what happened for stiff-soft-stiff clay; see the soil flow mechanisms in Figures 6a and 8c). For all cases with $s_{u1s}/s_{u2} = 0.11\sim 0.2$ and $t_1/D = 0.25$ ($s_{u1} = 2 + 2z$ kPa, $s_{u2} = 40$ kPa, $s_{u3} = 30 + 1(z - t_1 - t_2)$ kPa), the limiting squeezing depth (taken as the depth from where the spudcan started to sense the underlying stiff layer and consequently the resistance profile deviated from the one without the underlying layer) range is consistently about $\sim 0.18D$ from the surface of the stiff layer. A similar depth was reported by Hossain et al. (2011) from centrifuge model tests.

A comparison of the penetration resistance profiles for $s_{u2}/s_{u3s} = 1.33$ and 2.5 (see Figures 12a~12c) reflects that, in the middle layer, bearing response is higher and the curve is steeper for higher s_{u2}/s_{u3s} . This is yielded by the higher s_{u2} and also associated with the soil deformation extending deeper into the bottom layer of strength increasing with depth. Potential for rapid leg run over a depth of $1.4D$ exists only when the effect of higher strength ratio is supplemented by higher thickness ratio of $t_2/D = 0.75$ (see Figure 12c).

In most cases, soil backflow occurs in the bottom layer. The limiting cavity depth above the spudcan also increases as the strength ratio increases.

Effect of thickness ratios (t_1/D and t_2/D)

The discussion in this section primarily focuses on the influence of t_2/D , as in most case histories the 1st layer of this type of soil deposit is thin and soft and it barely affects the spudcan final penetration depth. The results are plotted in Figures 13a and 13b to explore the effect of thickness ratio for $t_2/D = 0.25, 0.5$ and 0.75 , with an identical $t_1/D = 0.25$ and $k_1D/s_{um} = 12$. The strength ratio at the interface between 2nd and 3rd layers are $s_{u2}/s_{u3s} = 1.33$ and 2.5, with the bottom layer non-homogeneity factor $k_3D/s_{u3s} = 0.4$.

Comparing Figures 13a and 13b, it can be seen that the effect of thickness ratio appears to be more pronounced for higher strength ratio s_{u2}/s_{u3s} , i.e. for higher value of s_{u2} as s_{u3s} is constant. For a strength ratio of $s_{u2}/s_{u3s} = 2.5$ ($s_{u1} = 2 + 2z$ kPa, $s_{u2} = 75$ kPa, $s_{u3} = 30 + 1(z - t_1 - t_2)$ kPa; see Figure 13b), a greater and steeper penetration resistance is mobilized in the thicker middle layer. The resistance profile remains somewhat constant through the stiff (2nd) layer and part of the bottom layer. A rapid leg run of about $1.4D$ would occur for this deposit with a sandwiched stiff layer of $t_2/D = 0.75$. The effect of thickness ratio and the form of the penetration resistance profiles discussed above are consistent with observations from the centrifuge tests (see Figure 8 of Hossain et al. 2011).

In the above analyses, the thickness of the top layer is $0.25D$. An analysis was also carried out doubling the thickness t_1 to $0.5D$ and the result is shown in Figure 13c ($s_{u1} = 2 + 2z$ kPa, $s_{u2} = 40$ kPa, $s_{u3} = 30 + 1(z - t_1 - t_2)$ kPa). The limiting squeezing depth is consistent of about $\sim 0.18D$, although the squeezing effect is not so prominent for $t_1/D = 0.5$ due to higher strength ratio $s_{u1s}/s_{u2} = 0.35$ (compared to 0.2 for $t_1/D = 0.25$). The trend is similar, but the profile lies below the curve for $t_1/D = 0.25$ due to delayed influence of the middle stiff layer.

The depth of soil backflow is also affected by the thickness ratios t_1/D and t_2/D . For $t_1/D = 0.25$, squeezing dominates the behavior shortly ($d/D = \sim 0.07$) after the spudcan's full diameter penetrates into the ground and, consequently, soil starts to flow back at deep penetration when the spudcan penetrates through the bottom layer. For $t_1/D = 0.5$, the thickness of the very soft top layer is sufficient to allow soil backflow to occur before squeezing and a downward soil deformation governing the behavior.

Effect of non-homogeneity factor of bottom layer (k_3D/s_{u3s})

To explore this influence of strength non-homogeneity k_3D/s_{u3s} comprehensively, analyses have been undertaken for a clay with $s_{u3s}/\gamma'D = 0.31$, but varying k_3D/s_{u3s} between 0 and 1.2 for thickness ratios of $t_1/D = 0.5$ and $t_2/D = 0.75$ (Group VI, Table 1). For all cases, the top layer with $k_1D/s_{um} = 7.2$ is underlain by the middle layer of uniform stiff clay with $s_{u2}/s_{u3s} = 2.5$. This range of parameters was chosen deliberately to illustrate the effect of the bottom layer strength gradient on punch-through potential.

Figure 14 shows the bearing resistance profiles. Severe punch-through with a negative post-peak response is evident in the bottom layer at $d/D = 1.4\sim 1.5$, following the initiation of soil backflow. Except for the sediment with a uniform bottom layer (i.e. $k_3D/s_{u3s} = 0$), the uncontrollable penetration continues until the rising capacity in the bottom layer comprising

non-uniform clay, re-establishes the failure load at $d/D = 1.7 \sim 2.2$. In the field, the spudcan would rapidly penetrate this distance.

The severity of punch-through reduces substantially as k_3D/s_{u3s} increases owing to the sharper increase in strength profile of the bottom layer. It is evident from Figure 14 that while severe punch-through happens for $k_3D/s_{u3s} = 0$, moderate ($0.7D$) or mild ($0.3D$) punch-through may occur as k_3D/s_{u3s} approaches 1.2. This is consistent with the effect reported by Hossain and Randolph (2010a, 2010b) on two-layer stiff-over-soft clays. They provided explanation for the effect of k_2D/s_{u2s} by the entirely different soil flow patterns for cases where all other parameters were identical but with different non-homogeneity factors (k_2D/s_{u2s}) of 0 and 0.6.

Punch-through failure occurs in the bottom layer with the initiation of soil backflow. The depth of backflow increases as k_3D/s_{u3s} reduces. From Figure 13c, for similar thickness ratios of $t_1/D = 0.5$ and $t_2/D = 0.75$, backflow occurs in the top layer. The delayed backflow in Figure 14 is due to lower k_1D/s_{um} of 7.2 (compared to 12 in Figure 13c).

Effect of accounting for strain rate and softening

Similar to stiff-soft-stiff clay, analyses were also carried out for strain softened, rate dependent soil ($\delta_{rem} = 0.3$, $\mu = 0.1$, $\xi_{95} = 12$, $\dot{\gamma}_{ref} = 1.5\%/h$). Two profiles are included in Figure 12c ($s_{u1} = 2 + 2z$ kPa, $s_{u2} = 40$ and 75 kPa, $s_{u3} = 30 + 1(z - t_1 - t_2)$ kPa) for comparison. Accounting for strain softening and rate dependency has insignificant effect at shallow penetration depth in the 1st soft layer. They exhibit with the potential for severe punch-through while miniature punch-through or rapid leg run occurs for ideal soil. Additionally, soil backflow occurs earlier in the 1st layer for spudcan penetration in strain softened, rate dependent soil. These are consistent with Figure 11.

Concluding Remarks

Continuous penetration of spudcan foundations from the surface of three-layer seabed sediments was investigated through 3D large deformation FE analyses using the CEL approach in Abaqus/Explicit. Two stratifications, including uniform stiff-soft-stiff and non-uniform clay with an interbedded stiff clay layer, commonly encountered in the field and identified as critical for potential punch-through failure, were considered. The effects of a practical range of normalized soil properties and layer geometries on the bearing response and soil flow mechanisms were discussed in the context of punch-through and rapid leg run (for stiff-over-soft) and squeezing (for the reverse). The following key conclusions can be drawn from the results presented in the paper.

1. For uniform stiff-soft-stiff clays, the presence of the bottom stiff clay layer within a short distance of $(t_1 + t_2) = 1D$ allowed for sensing that layer from the beginning of spudcan penetration in the top layer. This inhibited the potential for punch-through (apart from rapid leg run for $t_1/D = 0.75$ and $S_{u1s}/S_{u2} = 3$), which would occur for a higher thickness ratio of $(t_1 + t_2)/D = 1.5$ and for some identical, but two-layer stiff-over-soft clay deposits.
2. The soil plug trapped at the base of the advancing spudcan allowed for early sensing of the bottom stiff layer and hence enhanced the corresponding limiting squeezing depth.
3. The resistance profile from a spudcan continuous penetration on a three-layer stiff-soft-stiff clay deposit was significantly different from the combined profiles resulted from two separate analyses on 1st-2nd layer and 2nd-3rd layer even after an appropriate depth factor was accounted for. As such for using the ISO suggested bottom-up design approach, combining the punch-through (for stiff-over-soft layering) and squeezing (for

the reverse) criteria for two layer systems, at least appropriate plug heights and limiting squeezing depths are required to be considered.

4. For non-uniform clay with an interbedded stiff layer, spudcan bearing response in the top layer was dominated by squeezing, with a consistent limiting squeezing depth of about $\sim 0.18D$.

5. In contrast to ISO recommendation, the soft soils in between the advancing spudcan base and the stronger layer did not squeeze out completely instead some of the trapped material was forced into the stronger underlying layer.

6. The potential for punch-through failure or rapid leg run was shown to exist for $t_2/D \geq 0.5$ and $S_{u3s}/S_{u2} = 0.4$.

7. The non-uniform bottom layer with a positive strength gradient was shown to have a favorable influence on reducing punch-through distance. As k_3D/S_{u3s} increased from 0.4 to 1.2, the punch-through distance of $0.75D$ decreased to $0.3D$.

8. At very shallow penetration depths, the combined effect of strain softening and rate dependency had insignificant influence on spudcan penetration resistance. With the progress of penetration, the profiles for strain softened, rate dependent soils were diverged gradually from and became lower than that for ideal soil, leading to enhanced likelihood and severity for punch-through.

Acknowledgements

The first author is the recipient of the University of Western Australia SIRF and UIS scholarships. The 2nd author is an Australian Research Council (ARC) Discovery Early Career Researcher Award (DECRA) Fellow and is supported by the ARC Project DE140100903.

562 The research presented here was undertaken with support from the ARC and the industry
563 partner Keppel Offshore and Marine Pte Ltd, Singapore through the ARC Linkage Project
564 LP110100174. This work forms part of the activities of the Centre for Offshore Foundation
565 Systems (COFS), currently supported as a node of the Australian Research Council Centre of
566 Excellence for Geotechnical Science and Engineering and as a Centre of Excellence by the
567 Lloyd's Register Foundation. This support is gratefully acknowledged.

568 NOMENCLATURE

569	A	spudcan plan area at largest section
570	D	foundation diameter at largest section
571	d	penetration depth of spudcan base (lowest point at largest section) measured from
572		mudline
573	E	Young's modulus
574	H	open cavity depth after spudcan installation (taken as identical to backflow depth)
575	$h_{\min}$	minimum element size in numerical model
576	k_1	rate of increase of shear strength within 1 st layer
577	k_2	rate of increase of shear strength within 2 nd layer
578	k_3	rate of increase of shear strength within 3 rd layer
579	P	penetration resistance of spudcan foundation
580	q_{net}	net bearing pressure
581	$q_{\text{net}0}$	net bearing pressure in weightless soil
582	s_u	undrained shear strength of soil
583	s_{u1}	undrained shear strength of 1 st layer soil
584	s_{u1s}	undrained shear strength of 1 st layer soil at 1 st -2 nd layer interface
585	s_{u2}	undrained shear strength of 2 nd layer soil
586	s_{u2s}	undrained shear strength of 2 nd layer soil at 1 st -2 nd layer interface

587	s_{u3}	undrained shear strength of 3 rd layer soil
588	s_{u3s}	undrained shear strength of 3 rd layer soil at 2 nd -3 rd layer interface
589	s_{um}	undrained shear strength at mudline
590	t_1	thickness of 1 st layer soil
591	t_2	thickness of 2 nd layer soil
592	V	volume of embedded spudcan foundation including shaft
593	v_{sp}	spudcan penetration velocity in numerical simulation
594	z	depth below soil surface
595	γ'	effective unit weight of soil
596		

References

- Aust, T. (1997). *Accident to the mobile offshore drilling unit Maersk Victory on 16 November 1996*, Mines and Energy Resources, South Australia.
- Bennett and Associates, L. L. C, and Offshore Technology Development INC. (2005). "Jack up units: a technical primer for the offshore industry professional." <
<http://www.bennettoffshore.com/primer.html> >.
- Brennan, R., Diana, H., Stonor, R. W. P., Hoyle, M. J. R., Cheng, C. P., Martin, D., and Roper, R. (2006). "Installing jackups in punch-through-sensitive clays." *Proc., Offshore Technology Conf.*, Houston, OTC 18268.
- Brown, J. D., and Meyerhof, G. G. (1969). "Experimental study of bearing capacity in layered clays." *Proc., 7th Int. Conf. on Soil Mech. and Found. Eng.*, Mexico 2, 45–51.
- Carter, J. P., and Balaam, N. P. (1990). *AFENA user's manual*, Centre for Geotechnical Research, The University of Sydney, Sydney, Australia.
- Castleberry II, J. P., and Prebahrar, N. (1985). "Clay crusts of the Sunda Shelf - a hazard to jack-up operations." *Proc., 8th Southeast Asian Geotech. Conf.*, Kuala Lumpur, 40–48.
- Chan, N. H. C., Paisley, J. M., and Holloway, G. L. (2008). "Characterization of soils affected by rig emplacement and Swiss cheese operations - Natuna Sea, Indonesia, a case study." *Proc., 2nd Jack-up Asia Conf. and Exhibition*, Singapore.
- Club des Actions de Recherche sur les Ouvrages en Mer (CLAROM). (1993). *Design guides for offshore structures*, P. Le Tirant and C. Pérol, eds., Paris.
- Edwards, D. H., and Potts, D. M. (2004). "The bearing capacity of circular footing under "punch-through" failure." *Proc., 9th Int. Symp. on Numerical Models in Geomechanics*, Ottawa, 493–498.
- Einav, I., and Randolph, M. F. (2005). "Combining upper bound and strain path methods for evaluating penetration resistance." *Int. J. Numer. Methods in Eng.*, 63(14), 1991–2016.
- Erbrich, C. T. (2005). "Australian frontiers – Spudcans on the edge." *Proc., 1st Int. Symp. on Frontiers in Offshore Geotechnics (ISFOG)*, Taylor & Francis, London, 49–74.

- Handidjaja, P., Somehsa, P., and Manoj, M. (2004). “‘Swiss-cheese’ – A method of degrading soil crust and minimizing risk to punch through problem on the installation of mobile offshore drilling unit (MODU).” *Proc., 15th Southeast Asian Geotechnical Society Conf.*, Bangkok, 303–306.
- Hossain, M. S., and Randolph, M. F. (2010a). “Deep-penetrating spudcan foundations on layered clays: centrifuge tests.” *Géotechnique*, 60(3), 157–170.
- Hossain, M. S., and Randolph, M. F. (2010b). “Deep-penetrating spudcan foundations on layered clays: numerical analysis.” *Géotechnique*, 60(3), 171–184.
- Hossain, M. S., and Randolph, M. F. (2009a). “Effect of strain rate and strain softening on the penetration resistance of spudcan foundations on clay.” *Int. J. Geomech.*, 10.1061/(ASCE)1532-3641(2009)9:3(122), 122–132.
- Hossain, M. S., and Randolph, M. F. (2009b). “New mechanism-based design approach for spudcan foundations on single layer clay.” *J. Geotech. Geoenviron. Eng.*, 10.1061/(ASCE)GT.1943-5606.0000054, 1264–1274.
- Hossain, M. S., and Randolph, M. F. (2009c). “New mechanism-based design approach for spudcan foundations on stiff-over-soft clay.” *Proc., Offshore Technology Conf.*, Houston, OTC 19545.
- Hossain, M. S., Randolph, M. F., and Saunier, Y. N. (2011). “Spudcan deep penetration in multi-layered fine-grained soils.” *Int. J. Phys. Modelling in Geotech.*, 11(3), 100–115.
- Hu, Y., and Randolph, M. F. (1998). “A practical numerical approach for large deformation problems in soil.” *Int. J. Numer. Anal. Methods Geomech.*, 22(5), 327–350.
- InSafeJIP (2010). *Improved guidelines for the prediction of geotechnical performance of spudcan foundations during installation and removal of jack-up units*, RPS Energy, Abingdon, Oxfordshire, U.K.
- International Organization for Standardization (ISO). (2011). “Petroleum and Natural Gas Industries – Site Specific Assessment of Mobile Offshore Units – Part 1: Jack-ups.” *ISO 19905-1*, Geneva.

- 651 Jack, R. L., Hoyle, M. J. R., Hunt, R. J., and Smith, N. P. (2007). “Jack-up accident statistics:
652 Lots to learn!” *Proc., 11th Int. Conf. the Jack-up Plat Form Design, Construction and*
653 *Operation*, London.
- 654 Jack, R. L., Hoyle, M. J. R., Smith, N. P., and Hunt, R. J. (2013). “Jack-up accident statistics
655 – A further update.” *Proc., 11th Int. Conf. the Jack-up Plat Form Design, Construction*
656 *and Operation*, London.
- 657 Kostelnik, A., Guerra, M., Alford, J., Vazquez, J., and Zhong, J. (2007). “Jackup mobilization
658 in hazardous soils.” *SPE Drilling and Completion*, 22(1), 4–15.
- 659 Lu, Q., Randolph, M. F., Hu, Y., and Bugarski, I. C. (2004). “A numerical study of cone
660 penetration in clay.” *Géotechnique*, 54(4), 257–267.
- 661 Lunne, T., Berre, T., Andersen, K. H., Strandvik, S., and Sjursen, M. (2006). “Effects of
662 sample disturbance and consolidation procedures on measured shear strength of soft
663 marine Norwegian clays.” *Can. Geotech. J.*, 43(7), 726–750.
- 664 Martin, C. M., and Randolph, M. F. (2001). “Application of the lower and upper bound
665 theorems of plasticity to collapse of circular foundations.” *Proc., 10th Int. Conf. on*
666 *Computer Methods and Advance Geomechanics*, Vol. 2, Tucson, Ariz., 1417–1428.
- 667 Maung, U. M., and Ahmad, C. K. M. (2000). “Swiss cheesing to bring in a jack-up rig at
668 Anding location.” *Proc., IADC/SPE Asia Pacific Drilling Technology*, Kuala Lumpur,
669 IADC/SPE 62755.
- 670 Menzies, D., and Lopez, C. R. (2011). “Four atypical jack-up rig foundation case histories.”
671 *Proc., 13th Int. Conf. on the Jack-up Platform: Design, Construction and Operation*,
672 London.
- 673 Menzies, D., and Roper, R. (2008). “Comparison of jackup rig spudcan penetration methods
674 in clay.” *Proc., Offshore Technology Conf.*, Houston, OTC 19545.
- 675 Merifield, R. S., and Nguyen, V. Q. (2006). “Two- and three-dimensional bearing capacity
676 solutions for footing on two-layered clays.” *Geomech. Geoengng*, 1(2), 151–162.
- 677 Meyerhof, G. G., and Chaplin, T. K. (1953). “The compression and bearing capacity of
678 cohesive layers.” *Br. J. Appl. Phys.*, 4(1), 20–26.

- Qiu, G., and Grabe, J. (2011). "Explicit modelling of cone and strip footing penetration under drained and undrained conditions using a visco-hypoplastic model." *geotechnik*, 34(3), 205–217.
- Qiu, G., and Grabe, J. (2012). "Numerical investigation of bearing capacity due to spudcan penetration in sand overlying clay." *Can. Geotech. J.*, 49(12), 1393–1407.
- Qiu, G., and Henke, S. (2011). "Controlled installation of spudcan foundations on loose sand overlying weak clay." *Mar. Struct.*, 24(4), 528–550.
- Randolph, M. F. (2004). "Characterisation of soft sediments for offshore applications." *Proc., 2nd Int. Conf. on Site Characterization*, Porto, Portugal, Vol. 1, Millpress Science Publisher, Rotterdam, 209–231 (Keynote Lecture).
- Randolph, M. F., Cassidy, M. J., Gourvenec, S., and Erbrich, C. J. (2005). "Challenges of offshore geotechnical engineering. State of the Art paper." *Proc., Int. Conf. on Soil Mech. and Found. Eng.*, Int. Society for Soil Mechanics and Geotechnical Engineering, London, 123–176.
- Tho, K. K., Leung, C. F., Chow, Y. K., and Swaddiwudhipong, S. (2012). "Eulerian finite-element technique for analysis of jack-up spudcan penetration." *Int. J. Geomech.*, 12(1), 64–73.
- Thorne, C. P., Wang, C. X., and Carter, J. P. (2004). "Uplift capacity of rapidly loaded strip anchors in uniform strength clay." *Géotechnique*, 54(8), 507–517.
- Wang, C. X., and Carter, J. P. (2002). "Deep penetration of strip and circular footings into layered clays." *Int. J. Geomech.*, 2(2), 205–232.
- Wong, P. C., Templeton III, J. S., Purwana, O. A., Hofstede, H., Cassidy, M. J., Hossain, M. S., and Martin, C. M. (2012). "Foundation modeling and assessment in the new ISO standard 19905-1." *Proc., Offshore Technology Conf.*, Houston, OTC 23521.
- Young, A. G., Remmes, B. D., and Meyer, B. J. (1984). "Foundation Performance of offshore jack-up drilling rigs." *J. Geotech. Engrg.*, 10.1061/(ASCE)0733-9410(1984)110:7(841), 841–859.
- Zheng, J., Hossain, M. S., and Wang, D. (2013). "3D large deformation FE analysis of spudcan and cone penetration on three-layer clays." *Proc., 23rd Int. Offshore and*

- 708 *Polar Engineering Conf.*, Int. Society of Offshore and Polar Engineers, Cupertino,
709 CA, 453-460.mark
- 710 Zhou, M., Hossain, M. S., Hu, Y., and Liu, H. (2012). "Behaviour of ball penetrometer in
711 uniform single- and double-layer clays." *Géotechnique*, 63(8), 682–694.
- 712 Zhou, H., and Randolph, M. F. (2007). "Computational techniques and shear band
713 development for cylindrical and spherical penetrometers in strain-softening clay." *Int.*
714 *J. Geomech.*, 10.1061/(ASCE)1532-3641(2007)7:4(287), 287–295.
- 715 Zhou, H., and Randolph, M. F. (2009). "Resistance of full-flow penetrometers in rate-
716 dependent and strain-softening clay." *Géotechnique*, 59(2), 79–86.

717

Table 1. Summary of LDFF analyses performed

Analysis	S_{u1s}/S_{u2}	t_1/D	$k_1 D/S_{um}$	S_{u2}/S_{u3s}	t_2/D	$S_{u3s}/\gamma' D$	$k_3 D/S_{u3s}$	Spudcan base	Remarks
Group I	1, 2 and 10	1.0	0	-	Infinite	-	-	Smooth	Comparison with LDFF results of Wang and Carter (2002)
Group II	1.25, 1.67, 2.5 and 5	0.5	0	-	Infinite	-	-	Rough	Comparison with LDFF results of Hossain and Randolph (2010b)
Group III	0.2	0.38	3.6	2.6	0.58	0.1	0.81	Rough	Comparison with centrifuge test T6 (Hossain et al. 2011)
	1.82	0.42	0.6	0.33	0.5	0.38	0		Comparison with centrifuge test T8 (Hossain et al. 2011)
Group IV	1.25, 2 and 3	0.25, 0.5 and 0.75	0	0.53	0.25, 0.5 and 0.75	0.16 and 0.78	0	Rough	Parametric study for uniform stiff-soft-stiff clay profile
Group V	0.11, 0.2 and 0.35	0.25 and 0.5	12	1.33 and 2.5	0.25, 0.5 and 0.75	0.31	0.4	Rough	Parametric study for non-uniform clay with an interbedded stiff clay profile
Group VI	0.31	0.5	7.2	2.5	0.75	0.31	0, 0.4, 0.8 and 1.2	Rough	Exploration of the effect of bottom layer strength non-homogeneity

718

No. of Figure: 14

Fig. 1 Typical seabed strength profiles with potential for punch-through: (a) Uniform stiff-soft-stiff clay; (b) Non-uniform clay with an interbedded stiff layer

Fig. 2 Schematic diagram of embedded spudcan foundation showing idealized open cavity

Fig. 3 Typical mesh used in CEL analyses

Fig. 4 Comparison between results of LDFE analysis using CEL and RITSS: (a) Circular footing on weightless uniform stiff-over-soft clay (Group I, Table 1); (b) Spudcan on weighty uniform stiff-over-soft clay (Group II, Table 1)

Fig. 5 Comparison between results of LDFE analysis and centrifuge test data (Group III, Table 1): (a) Test T6 ($s_{u1} = s_{u3} = 2 + 0.6z$ kPa, $s_{u2} = 23$ kPa); (b) Test T8 ($s_{u1} = 16 + 0.8z$ kPa, $s_{u2} = 11$ kPa, $s_{u3} = 33$ kPa)

Fig. 6 Soil failure mechanisms during spudcan penetration: (a) early squeezing due to trapped soil plug ($d/D = 0.48$, $s_{u1} = 120$ kPa, $s_{u2} = 40$ kPa, $s_{u3} = 75$ kPa, $t_1/D = t_2/D = 0.5$); (b) Layered soil plug comprising soft soil from top layer ($d/D = 0.55$, $s_{u1} = 2 + 2z$ kPa, $s_{u2} = 75$ kPa, $s_{u3} = 30 + 1(z - t_1 - t_2)$ kPa, $t_1/D = t_2/D = 0.25$)

Fig. 7 Effect of strength ratio (s_{u1s}/s_{u2}) on bearing behavior: uniform stiff-soft-stiff clay ($s_{u3s}/\gamma'D = 0.78$, rough; Group IV, Table 1): (a) $t_1/D = 0.75$, $t_2/D = 0.25$; (b) $t_1/D = 0.5$, $t_2/D = 0.5$; (c) $t_1/D = 0.25$, $t_2/D = 0.75$

Fig. 8 Effect of thickness ratios (t_1/D and t_2/D) on bearing behavior for $(t_1 + t_2)/D = 1.0$: uniform stiff-soft-stiff clay ($s_{u3s}/\gamma'D = 0.78$, rough; Group IV, Table 1): (a) $s_{u1s}/s_{u2} = 3$; (b) $s_{u1s}/s_{u2} = 2$; (c) $s_{u1s}/s_{u2} = 1.25$

Fig. 9 Effect of thicker top and middle layers on bearing behaviour: uniform stiff-soft-stiff clay ($s_{u1s}/s_{u2} = 3$, $s_{u3s}/\gamma'D = 0.78$, rough; Group IV, Table 1)

Fig. 10 Effect of normalized strength ($s_{u3s}/\gamma'D$) on bearing behavior: uniform stiff-soft-stiff clay ($s_{u1s}/s_{u2} = 3$, $s_{u2}/s_{u3s} = 0.53$, $t_1/D = t_2/D = 0.75$, rough; Group IV, Table 1)

Fig. 11 Comparison of results with and without the effects of strain softening and rate dependency ($s_{u3s}/\gamma'D = 0.78$, rough; Group IV, Table 1)

Fig. 12 Effect of strength ratio (s_{u2}/s_{u3s}) on bearing behavior: non-uniform clay with an interbedded stiff layer ($s_{u3s}/\gamma'D = 0.31$, $k_1D/s_{um} = 12$, $k_3D/s_{u3s} = 0.4$, rough; Group V, Table 1): (a) $t_1/D = 0.25$, $t_2/D = 0.25$; (b) $t_1/D = 0.25$, $t_2/D = 0.5$; (c) $t_1/D = 0.25$, $t_2/D = 0.75$

Fig. 13 Effect of thickness ratios (t_1/D and t_2/D) on bearing behavior: non-uniform clay with an interbedded stiff layer ($s_{u3s}/\gamma'D = 0.31$, $k_1D/s_{um} = 12$, $k_3D/s_{u3s} = 0.4$, rough; Group V, Table 1): (a) $s_{u2}/s_{u3s} = 1.33$, $t_1/D = 0.25$; (b) $s_{u2}/s_{u3s} = 2.5$, $t_1/D = 0.25$; (c) $s_{u2}/s_{u3s} = 1.33$, $t_2/D = 0.75$

Fig. 14 Effect of bottom layer non-homogeneity (k_3D/s_{u3s}) on bearing behavior: non-uniform clay with an interbedded stiff layer ($s_{u3s}/\gamma'D = 0.31$, $t_1/D = 0.5$, $t_2/D = 0.75$, $k_1D/s_{um} = 7.2$, rough; Group VI, Table 1)