Penetration Response of Spudcans in Layered Sands

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ABSTRACT

The behaviour of spudcan foundations during the installation and preloading in two-layer sand sediments was investigated through large deformation finite element (LDFE) analyses. The LDFE analyses were carried out using the coupled Eulerian-Lagrangian approach, modifying Mohr-Coulomb soil model to capture hardening and subsequent softening effects of sand. Parametric analyses were undertaken varying the top layer thickness, relative density of sand and spudcan diameter. Both loose to medium dense-over-dense and dense-over-loose to medium dense sand deposits were explored. The results showed that, for the investigated relatively thin top layer thickness of ≤ 5 m, spudcan behaviour was dictated by the bottom sand layer with a minimal influence of the top layer. For assessing the penetration resistance profile in two-layer sands, the performance of the ISO, SNAME, InSafeJIP, and other existing theoretical design methods were evaluated.

Keywords: Spudcan; Penetration resistance, Large deformation finite element, Two-layer sands

41 Nomenclature

42	A	spudcan plan area at largest section
43	D	spudcan (or footing) diameter at largest section
44	d	penetration depth of spudcan (or footing) base
45	d_q	depth factor for sand
46	d_γ	depth factor on surcharge for sand
47	F_{mob}	mobilisation factor
48	I_D	relative density of sand
49	I_{Db}	relative density of bottom sand layer
50	I_{Dt}	relative density of top sand layer
51	K_s	coefficient of punching shear resistance
52	N_q, N_γ	dimensionless bearing capacity factor
53	$N_{qb}, N_{\gamma b}$	dimensionless bearing capacity factor for bottom sand layer
54	$N_{qt}, N_{\gamma t}$	dimensionless bearing capacity factor for top sand layer
55	Q	penetration resistance
56	Q_b	bearing resistance of bottom sand layer
57	Q_t	bearing resistance of top sand layer
58	q	bearing pressure
59	q_u	ultimate bearing capacity
60	S_s	shape factor for punching shear resistance on cylindrical surface
61	s_q	shape factor for bearing capacity factor N_q
62	s_γ	shape factor for bearing capacity factor N_γ
63	t	thickness of top sand layer

64	t_f	depth of failure surface beneath footing in loose-over-dense sand deposits
65	V_c	volume of spudcan
66	V_{soil}	volume of backfill soil
67	$\Delta\epsilon_1, \Delta\epsilon_2, \Delta\epsilon_3$	incremental principal plastic strains
68	γ'	effective unit weight of soil
69	γ_b'	effective unit weight of bottom sand layer
70	γ_t'	effective unit weight of top sand layer
71	ξ	accumulated plastic shear strain
72	ξ_{cv}	threshold plastic shear strain corresponding to ϕ_{cv}
73	ξ_p	threshold plastic shear strain corresponding to ϕ_p
74	ϕ	internal friction angle
75	ϕ_b	friction angle of bottom sand layer
76	ϕ_t	friction angle of top sand layer
77	ϕ_{cv}	critical state friction angle
78	ϕ_{ini}	initial value of friction angle
79	ϕ_p	peak value of friction angle
80	ψ	dilation angle
81	ψ_p	peak dilation angle

82

83

84

1 INTRODUCTION

Mobile jack-up rigs are widely used for most offshore drilling operations in water depths up to around 150 m [1, 2, 3, 4]. A typical jack-up rig is comprised of a buoyant triangular hull platform and normally three independent truss legs. Each leg is vertically retractable by a rack-and-pinion jacking system and rests on a large 6 ~ 22 m diameter spudcan. Jack-ups can be installed on a site without intervention of any auxiliary equipment. After lowering their legs to the seabed, empty ballast tanks in the hull are filled with sea water and the footings are then pre-loaded. This is to ensure that the footings have sufficient reserve capacity from the seabed in any extreme storm.

Layered sand deposits (dense-over-loose and the reverse) result from various geological processes, including sand channelling and evolving depositional environments associated with changing sea level. These sediments have been especially critical for the assessment of spudcan penetration for the heavy lift jack-up vessel, which is frequently used for construction of offshore windfarm in the North Sea [5]. This is because if a spudcan is partially penetrated in sand, then the contact area defines the moment capacity and rotational stiffness of the foundation. The depth of a spudcan penetration anticipated in sand has also an important factor for determining whether scour could be problematic. The bearing capacity profile with depth is critical in determining the additional settlements that can occur if the forces on the spudcan due to an extreme event are outside the yield interaction surface computed for the spudcan at the penetration achieved during installation. Such settlements often result in a gain in capacity through expansion of the yield interaction (i.e. Step 3a check in ISO 19905-1; [6]). Under-prediction or over-prediction of bearing capacity in sand can both lead to unconservative outcomes. Layered sand sediments are also prevalent in the Gulf of Thailand, South China Sea, Arabian Gulf, North Sea, offshore India [7].

A very limited number of investigations have been carried out on foundation response in layered sand deposits through 1g model tests on the laboratory floor, mostly limited to strip or circular footings resting on the surface of the soil, with the assumption that the displacement of the footing prior to ultimate load is very small. Hanna [8, 9] conducted model tests using a circular footing (of diameter $D = 50$ mm and thickness 38 mm) with a rough base sitting on the surface of two-layered soils. Air-dried medium to coarse angular silica sand was used. The dense sand layers of $I_D = 69.1\%$ ($\phi = 47.7^\circ$) were produced by raining the sand, and the loose sand layers of $I_D = 21.8\%$ ($\phi = 34^\circ$) were produced by pouring the sand slowly from a very low height. Both dense-over-loose and loose-over-dense deposits were tested, covering the ratio of the top layer thickness below the foundation base to the foundation diameter, $(t - d)/D = 0.0, 0.5, 1.0, 2.0, 2.5$ and ∞ ; and the foundation's normalised pre-embedment depth of $d/D = 0.0, 0.5$, and 1.0 (where t is the thickness of the top sand layer; d is the embedment depth of the foundation; D is the foundation diameter). For the dense-over-loose sand deposits, the ultimate bearing capacity increased with increasing $(t - d)/D$ and d/D , and for the reverse deposits, that reduced rapidly with increasing $(t - d)/D$. Meyerhof & Hanna [10] also presented some 1g test results on a rough circular ($D = 75$ mm). Tests were carried out on both dense ($I_D = 69\%$)-over-loose ($I_D = 22\%$) silica sand deposits and on the reverse, encompassing $(t - d)/D = 0.0 \sim \infty$; and $d/D = 0.0 \sim 1.0$. The results are presented in Figure 1.

For assessing ultimate vertical bearing capacity of a circular footing on loose-over-dense sand deposits, Meyerhof [11] and Meyerhof & Hanna [10] proposed a theoretical equation as

$$Q = Q_t + (Q_b - Q_t) \left(1 - \frac{t-d}{t_f} \right) \geq Q_t \quad (1)$$

where Q_b and Q_t are the ultimate vertical bearing capacity of footings on a thick bed of the bottom and top sand, respectively, and can be calculated according to

$$Q_b = A(0.5\gamma'_b DN_{\gamma b} + \gamma'_t tN_{qb}) \quad (2)$$

$$Q_t = A(0.5\gamma'_t DN_{\gamma t} + \gamma'_t dN_{qt}) \quad (3)$$

t_f is the depth of failure surface beneath the footing in a thick deposit of loose sand material, with $t_f = 1.0D$ being suggested following Meyerhof [12].

For a circular footing on dense-over-loose sand deposits, the ultimate vertical bearing capacity can be calculated as [8, 10]

$$Q = Q_b + 2A\gamma'_t(t-d)^2 \left(1 + \frac{2d}{t-d}\right) S_s K_s \frac{\tan\phi_t}{D} - A\gamma'_t(t-d) \leq Q_t \quad (4)$$

where S_s is the shape factor for punching shear resistance on cylindrical surface (~ 1.0). Punching shear coefficient K_s depends on the strength of both the top and bottom layer, which can be derived from the proposed chart [8]. The definitions of the other terms are given under notation. Estimated profiles using Equations 1 and 4 have been added in Figure 1, showing a reasonable agreement.

A spudcan penetrates continually from the soil surface involving large deformation of the adjacent soils. As such, the theoretical solutions just discussed may not be applicable for assessing spudcan's continuous penetration resistance profiles, as discussed later.

The purpose of this paper is to investigate continuously penetrating spudcans in two-layer sand deposits through three dimensional large deformation finite element (3D LDFE) analyses. Hardening and subsequent softening behaviour of sand are accounted for since both can have a significant effect on penetration resistance profiles and soil failure mechanisms. An extensive parametric investigation was undertaken, varying the parameters related to the

top layer thickness, relative density of sand and spudcan size. Various design approaches were also compared with the results obtained for the layered sand deposits.

2 NUMERICAL ANALYSIS

2.1 Geometry and parameters

This study has considered a circular spudcan. The shape was selected similar to the spudcans of the ‘Marathon LeTourneau Design, Class 82-SDC’ jack-up rig, as illustrated by Menzies & Roper [13]. The diameter of the spudcans (D) for the heavy lift jack-up vessel (used for installing wind turbines and for installing and decommissioning oil and gas infrastructures) ranges from 5 to 8 m; and that of the spudcans for the oil and gas drilling rigs varies between 12 and 20 m. Layered sand deposits have been especially critical for the assessment of spudcan penetration for the heavy lift jack-up vessel in the North Sea, first the analyses have been carried out considering $D = 6$ m, and then for $D = 14$ and 20 m. Figure 2 shows a schematic diagram of a spudcan installed in two-layered sand deposits. The top sand layer of thickness (t) with uniform initial relative density (I_D) is underlain by another sand layer of (nominally) infinite depth. The relative density and hence effective unit weight of each sand layer were varied for loose to dense ($I_D = 15 \sim 100\%$; $\gamma' = 9 \sim 10 \text{ kN/m}^3$). The critical state friction angle (ϕ_{cv}) was taken as 32° for the sand [14, 15, 16]. The thickness of the top sand layer was varied as $t = 1 \sim 5$ m. The selected parameters for this study are assembled in Table 1, encompassing most cases of practical interest.

2.2 Analysis details

3D LDFE analyses were carried out using the coupled Eulerian-Lagrangian (CEL) approach in the commercial FE package ABAQUS/Explicit [17]. Qiu et al. [18], Tho et al. [19], Hu et al. [16, 20], Zheng et al. [21] investigated spudcan penetration behaviour using the CEL

approach and provided confidence to its applicability to solve problems involving large deformations.

Considering the symmetry of the problem, only a quarter spudcan and soil domain were modelled. A typical mesh is shown in Figure 3. The Eulerian soil mesh comprised 8-noded linear brick elements (termed EC3D8R in ABAQUS) with reduced integration. As justified from preliminary convergence studies (e.g. Hu et al. [16] and Zheng et al. [21]), the radius of the soil domain was 3.25D and the typical soil element size along the trajectory of the spudcan was adopted as 0.025D ('Fine mesh zone' in Figure 3). The spudcan was modelled as a rigid body and controlled not to tilt and remain vertical during the penetration process. The penetration velocity of the spudcan was set as 0.2 m/s into the soil.

The penetration of spudcans in two-layered sand is completed under drained conditions. Tho et al. [19] and Hu et al. [20] used the Mohr-Coulomb (MC) model to represent the mechanical response of sand with a spudcan continuous penetration. However, in its most basic form the MC model does not consider the effect of hardening-softening shear strength or variation of the dilatancy of sand, which are particularly critical for modelling the response of dense sand. To capture hardening and subsequent softening effects, the soil was modelled by the modified Mohr-Coulomb (MMC) model proposed by Hu et al. [16] and Zheng et al. [21]. Based on the traditional MC model, MMC was modified by varying the internal friction angle (ϕ) and dilation angle (ψ) with respect to the accumulated plastic shear strain (ξ) as shown schematically in Figure 4. It was assumed that the friction angle increases linearly from an initial value, ϕ_{ini} , to a peak value ϕ_p , before reducing linearly to ϕ_{cv} when the critical state is approached. The threshold plastic shear strains corresponding to peak friction angle and critical state are denoted as ξ_p and ξ_{cv} , respectively. The dilation angle remains zero when $\xi \leq 1\%$ and then increases quickly to a peak value, ψ_p , at $\xi = 1.2\%$. The dilation angle then

remains at ψ_p until ξ_p , followed by a linear reduction back to zero by ξ_{cv} . The threshold plastic shear strains of $\xi_p = 4\%$ and $\xi_{cv} = 10\%$ were selected based on the triaxial compression tests of super-fine silica sand by Pucker et al. [22]. The incremental plastic shear strain during each incremental step was calculated as

$$\Delta\xi = \sqrt{\frac{2[(\Delta\varepsilon_1 - \Delta\varepsilon_2)^2 + (\Delta\varepsilon_2 - \Delta\varepsilon_3)^2 + (\Delta\varepsilon_3 - \Delta\varepsilon_1)^2]}{3}} \quad (5)$$

where $\Delta\varepsilon_1$, $\Delta\varepsilon_2$ and $\Delta\varepsilon_3$ are incremental principal plastic strains measured from the start to the end of the current step. Then the friction and dilation angles at each integration point are updated through the relationships (see Figure 4) for the next step. The updated friction and dilation angles remain constant during the next step.

The initial friction angle ϕ_{ini} was taken as equal to ϕ_{cv} , which is the minimum value that can be operative when sand is in an initial state of shearing [14, 16]. The peak friction (ϕ_p) and dilation (ψ_p) angles were taken as a function of initial relative density (I_D) by Bolton [23]. The frictional spudcan-sand contact was described by the Coulomb friction law, with the coefficient of friction equal to $0.5\tan\phi_{cv}$, used by SNAME [24], Qiu & Henke [25] and Hu et al. [16]. Typical computation times on a high performance workstation with 12 CPU cores were about 35 hours for a spudcan penetration.

2.3 Validation against centrifuge test data

As of concern, there are no measured data from spudcan or any footing continuous penetration in sand-over-sand deposits, as also noted in introduction. Therefore, the numerical model was validated against two different sets of centrifuge test data on single layer sand. The tests were performed on the super-fine silica sand commonly used for testing at the University of Western Australia, and the properties of which used for the numerical analyses.

Bienen et al. [26] carried out a test at 200g on a flat circular footing of equivalent prototype diameter 12 m. The base of the flat footing was roughened by gluing sand onto it. The relative density of the dry sand was $I_D = 45\%$. An LDFE analysis was carried out modelling the footing base as rough. White et al. [14] conducted a test on a 4.8 m (at 80g) diameter conical (base envelope angle 150°) circular footing in a dry sand deposit of relative density, $I_D = 54\%$. Corresponding analyses were carried out considering interface friction coefficient of 0.5, as suggested by White et al. [14]. For all these analyses, peak friction (ϕ_p) and dilation angles (ψ_p) were calculated by using Bolton's correlations as $\phi_p = 35.75^\circ$, $\psi_p = 7.81^\circ$ for $I_D = 45\%$; and $\phi_p = 37.1^\circ$, $\psi_p = 10.6^\circ$ for $I_D = 54\%$).

The validation data are presented in Figure 5 in terms of the bearing pressure, q ($= Q/A$; where Q is the penetration resistance and A is the largest plan area of the footing), as a function of normalised penetration depth (d/D ; where d is the penetration depth of footing base). It is seen that the bearing pressure profiles from the LDFE simulations show a good agreement with the measured data. Note, sands used in these validation cases are mostly medium dense sand ($I_D = 45\% \sim 54\%$). Although load-penetration curves on very dense sand are available, those were not deemed reliable or not designed to obtain full-load-penetration curves, as summarised in Puker et al. [22]. As such, the available data are insufficient to assess for the higher relative density in a single layer sand. However, Hu et al. [16] and Zheng et al. [21] used the identical sand model, and carried out a series of validation analyses against centrifuge tests for spudcan penetration analyses in very dense sand-over-clay or inter-bedded very dense sand layer deposits. These validation also exercises have confirmed the capability and accuracy of the current numerical model in assessing spudcan penetration resistance in sand deposits.

3 BEHAVIOUR OF SPUDCAN FOUNDATIONS IN TWO-LAYERED SAND

An extensive parametric study was carried out varying (a) thickness of the top sand layer ($t = 1$ to 5 m); (b) relative density ratio of two layers ($I_{Dt}/I_{Db} = 0.15$ to 6.67); and (c) spudcan diameter ($D = 6$ to 20 m), as assembled in Table 1. All the input parameters of MMC model for sand were also summarised in Table 2.

3.1 Penetration resistance profiles and soil failure mechanisms

The resistance profiles and corresponding soil failure mechanisms during penetration of the spudcan with different relative density ratios of $I_{Dt}/I_{Db} = 15\%/100\% = 0.15$ (loose-over-dense sand) and $100\%/15\% = 6.67$ (dense-over-loose sand) are shown in Figures 6 and 7, respectively. The spudcan diameter ($D = 6$ m) and top layer thickness ($t = 5$ m) were fixed (in Group I, Table 1). The results from spudcan penetration in the corresponding single layered sand ($I_D = 15\%$ and 100%) and the estimated profiles using Equation 1 and Equation 4 are also included for comparison. The soil failure mechanisms display 4 different penetration stages from the soil surface. For loose-over-dense sand ($I_{Dt} = 15\%$ and $I_{Db} = 100\%$), from the beginning of penetration ($d = 0$ m; Figure 6b), the soil deformation is restricted in the upper loose sand layer by the bottom dense sand layer. A further advancement of the spudcan ($d = 2.5$ m; Figure 6c) results in squeezing of soil in the top layer. At the layer interface ($d = 5.0$ m; Figure 6d), interestingly, it can be seen that a thin loose sand plug is trapped at the base of the spudcan and pushed into the bottom dense sand layer. In the bottom layer ($d = 8.8$ m; Figure 6e), the loose sand plug is nearly diminished and a sand column is formed above the advancing spudcan. As such, the corresponding penetration resistance profile detaches from that the one for a single layer loose sand from the beginning of penetration, then increases slowly, finally rises sharply and merges with the one on single layer dense sand. The

estimated profile using Equation 1 significantly overestimates the computed penetration resistance in the top loose sand layer.

For dense-over-loose sand ($I_{Dt} = 100\%$ and $I_{Db} = 15\%$; Figure 7), the soil deformation is predominantly directed to the lower layer from the onset of penetration ($d = 0$ m). A thicker truncated cone of dense sand is trapped at the base of the spudcan and forced into the bottom loose sand layer (see Figures 6c, d). However, with further penetration ($d = 10$ m; Figure 7e), the thickness of the sand plug keeps reducing and a column of sand is formed above the spudcan. As such, the corresponding penetration resistance profile separates from that on single layer dense sand from the beginning of penetration, then keeps increasing very slowly, with the rate decreases with penetration depth. In contrast, the estimated profile using Equation 4 shows a different trend in the top dense sand layer – increases and then decreases forming a peak i.e. shows the likelihood of punch-through failure. This difference can be explained with the evolution of the friction angle, as shown in Figure 8[#], illustrating distributions of the mobilised friction angle in the top layer sand layer at different penetration depths. When the spudcan shoulder is fully embedded in the sand (see Figure 8b), only the sand beneath the shoulder is sheared at the critical state. With further penetration, the friction angle of the top dense sand in the outer region is quickly mobilised towards the peak value. At the depth of the original layer interface (see Figure 8d), the majority of the sand around the advancing spudcan has already reached the critical state, which is identical to friction angle of the underlying sand layer ($\phi_p = \phi_{cv} = 32^\circ$). This leads to stabilise the penetration resistance profile in bottom sand layer.

3.2 Effect of relative density

The effect of the relative density (I_{Dt} and I_{Db}) was investigated, varying the relative density combinations in two-layer sands, but keeping the other parameters as constant ($D = 6$ m and t

= 5 m; Group I, Table 1). Figure 9 shows the penetration resistance profiles and soil flow mechanisms. Overall, the penetration resistance (Q) increases along the base penetration depth (d). Interestingly, the bottom sand layer has significant influence on the penetration resistance (Q), while the upper sand layer shows a marginal effect. For instance, if the relative density of the bottom sand layer is fixed as $I_{Db} = 100\%$ (e.g. very dense sand), the profiles of penetration resistance with various top sand layer's relative densities ($I_{Dt} = 15, 35$ and 65%) converge on an identical load-penetration curve. Where the spudcan base is at the original level of the layer interface ($d = t$), the amounts of trapped plug are also almost identical (see insets of Figure 9). In the reverse ($I_{Dt} = 100\%$; $I_{Db} = 15, 35$ and 65%), the penetration resistance (Q) increases with increasing the relative density of the bottom sand layer (I_{Db}). The trapped soil plugs are forced down into the bottom layer, but the amount of the trapped soil decreases with increasing I_{Db} . As noted previously, these soil plugs squeeze out with further penetration. Again, it confirms that the bottom soil is the dominant factor for the spudcan penetration resistance in sand overlying sand.

3.3 Effect of thickness of top sand layer

To explore the effect of thickness of the top sand layer on the form of load-penetration response, the profiles of $t = 1, 3$ and 5 m are plotted in Figure 10, but keeping $D = 6$ m, with two extreme relative density combinations of $I_{Dt}/I_{Db} = 15\%/100\%$ and $100\%/15\%$ (in Group I, Table 1). For dense-over-loose sands ($I_{Dt} = 100\%$ and $I_{Db} = 15\%$), the initial penetration resistance slightly increases with increasing the top dense layer thickness (t). It is because more soil from the top sand layer was trapped underneath the spudcan with increasing t (see insets in Figure 10). However, this discrepancy reduces gradually with further penetration. For loose-over-dense sand ($I_{Dt} = 15\%$ and $I_{Db} = 100\%$), the top loose sand layer squeezes out later with increasing t . This leads to a lower penetration resistance at the initial penetration

stage. With further penetration, all the curves tend to approach the steady resistance corresponding to the bottom sand layer.

3.4 Effect of spudcan size and stress level

In LDFE analysis results presented so far, the spudcan diameter was kept constant as $D = 6$ m, representing the spudcan size of the heavy lift jack-up vessel used for installing wind turbines and for installing and decommissioning oil and gas infrastructures. To investigate the effect of spudcan size and to cover the spudcan size of mobile offshore drilling rigs, analyses were also carried out varying D to 14 m and 20 m (Groups II and III, Table 1). The results are presented in Figure 11. The penetration resistance Q was normalised by the spudcan plan area ($q = Q/A$), and non-dimensionalised according to $q/\gamma'D$ [$Q/(\pi\gamma'D^3/8)$]; and presented in Figures 11a and 11b, respectively against the normalised penetration depth, d/D . As the thickness of the top layer (t) was initially fixed at 1, 3 and 5 m, the relative thickness ratio (t/D) ranged from 0.05 to 0.833 (see Table 1). From Figure 11a, the bearing pressure increases with spudcan size regardless of t/D . This increasing rate is more profound when the bottom sand is denser. For instance, for loose-over-dense sand ($I_{Dt} = 15\%$ and $I_{Db} = 100\%$), q increases by 173 ~ 239% at $d/D = 1.0$ as D increases from 6 to 20 m, but that difference reduces to 150 ~ 172% for dense-over-loose sand ($I_{Dt} = 100\%$ and $I_{Db} = 15\%$). With increasing spudcan size, the less soil was trapped beneath the spudcan due to the reduction of t/D , and hence the squeezing mechanism occurs earlier (see insets in Figure 11a). An identical trend of increasing bearing pressure with increasing spudcan diameter can be observed on the results plotted by White et al. [14] on single layer dense ($I_D = 78\%$) and medium dense sand ($I_D = 54\%$) deposits. They carried out centrifuge tests on flat and conical based footings of diameter 0.6 m, 2.4 m and 4.8 m. The stress level effect can be quantified from the dimensionless load in Figure 11b. At any normalised penetration depth, the stress exerted on the soil under 20 m diameter spudcan is

higher than under a 6 m diameter spudcan, leading to a softer response for larger diameter. This is consistent with [14, 26, 27, 28, 29].

4 COMPARISONS WITH DESIGN APPROACHES

The results of the LDFE analyses have been compared with the calculations by various design guidelines, such as SNAME [24], InSafeJIP [7] and ISO [6]. The estimations using Equations 1 and 4 have not been included here as they showed significant overestimation or different trend (Figures 6a and 7a). Based on the results from parametric studies, the bottom layer sand dominated the behaviour of spudcan regardless of the top layer. Therefore, for the calculations using the methods in the design guidelines, the layered soil was treated as a single layer sand with the bottom layer properties. The bearing capacity expressions are as follows:

$$Q = \gamma' A (0.5DN_{\gamma}d_{\gamma}s_{\gamma} + dN_q s_q d_q) + \gamma'(V_C - V_{soil}) \quad \text{from SNAME [24]} \quad (6)$$

$$Q = \gamma' AF_{mob} (0.5DN_{\gamma}d_{\gamma} + dN_q s_q d_q) + \gamma'(V_C - V_{soil}) \quad \text{from InSafeJIP [7]} \quad (7)$$

$$Q = \gamma' A (0.5DN_{\gamma}d_{\gamma} + dN_q d_q) + \gamma'(V_C - V_{soil}) \quad \text{from ISO [6]} \quad (8)$$

The terms used in the above expressions are defined under notation. The bearing capacity in sand mainly depends on the friction angle. In sands, significant displacements are generally required to mobilise the theoretically available bearing capacity. If a calculation is based on realistic values of the mobilised or operational friction angle then, for a penetration process such as that of an approximately conical spudcan, the soil resistance will be significantly overestimated as at any stage of penetration the deformations in the soil will be insufficient to have mobilised the full strength of the sand throughout the relevant region [14, 26]. In the SNAME guideline [24], this problem was dealt with by artificially reducing the friction angle

by 5° for the bearing capacity calculation. The InSafeJIP guideline [7] also suggested a similar approach with a mobilisation factor (F_{mob}) of 0.25 to 0.5 (see Equation 7).

Figure 12 shows a comparison exercise in terms of load-penetration curve for a spudcan of $D = 20$ m penetrating in a dense-over-loose sand deposit ($I_{Dt} = 100\%$ and $I_{Db} = 15\%$; $t/D = 0.05\sim 0.25$). All the input parameters were summarised in Table 3. It can be seen that both approaches using a reduction in friction angle or applying a mobilisation factor estimate the behaviour of spudcan well in two-layer sands. In particular, $F_{mob} = 0.38$ provides an excellent fit to the LDFE results. However, Equation 8, which is based on realistic values of the peak friction angle (ISO, [6]), overestimates the overall penetration resistance profile.

5 CONCLUDING REMARKS

This paper has reported results of LDFE analyses investigating the penetration resistance profiles and associated soil failure mechanisms of spudcan foundations during deep penetration through two-layer loose to medium dense-over-dense and dense-over-loose to medium dense sand deposits. The LDFE analyses were undertaken with the spudcan foundation penetrated continuously from the soil surface. A modified Mohr-Coulomb soil model recently extended to capture hardening and subsequent softening effects of sand was adopted. The parametric study covered a practical range of the thickness of the top layer, relative density of the sand layers, and diameter of the spudcan. The following key conclusions can be drawn from the results presented in the paper.

1. For loose to medium dense-over-dense sand deposits with $t \leq 5$ m, a squeezing mechanism dominated the spudcan behaviour in the top layer, and hence the corresponding penetration resistance profile separated from the response on single

layer weaker sand from the beginning of penetration, and eventually merged with the one on single layer dense sand.

2. For dense-over-loose to medium dense sand deposits with $t \leq 5$ m, a dense sand plug was trapped at the base of the spudcan in the top dense sand layer and was forced into the bottom weaker sand layer, with the thickness of the plug kept reducing and eventually diminished in the weaker layer. As such, the corresponding penetration resistance profile detached from that on single layer dense sand from the beginning of penetration and then kept increasing very slowly with penetration depth. No punch-through type profile with a sharp drop forming a local peak in the dense sand layer was observed.

3. In general, for the investigated thickness of the top layer ($t \leq 5$ m), the bottom sand layer dominated the behaviour, with the penetration resistance increased with increasing the relative density of the bottom sand layer, and the influence of the thickness of the top layer was minimal.

4. The bearing pressure increased with increasing spudcan diameter regardless of the thickness of the top layer, and the influence was more profound for denser bottom layer. The reverse trend is true for the dimensionless load.

5. By comparing with the exiting design methods, it was found that the method suggested by SNAME [24] for single layer sand with a reduced friction angle, or the method recommended by InSafeJIP [7] for single layer sand with a mobilisation factor provided reasonable estimate to the LDFE results.

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Table 1. Summary of 3D LDFE analyses performed

Group	D (m)	Top sand layer			Bottom sand layer	Remark
		t (m)	t/D	I _{Dt} (%)	I _{Db} (%)	
I	6	1	0.167	15	100	Small diameter t/D = 0.167~0.833
				35		
				65		
				100	15	
					35	
					65	
		3	0.5	15	100	
				35		
				65		
				100	15	
					35	
					65	
		5	0.833	15	100	
				35		
				65		
				100	15	
					35	
					65	
II	14	1	0.071	15	100	Medium diameter t/D = 0.071~0.357
				100	15	
		3	0.214	15	100	
				100	15	
		5	0.357	15	100	
				100	15	
III	20	1	0.05	15	100	Large diameter t/D = 0.05~0.25
				100	15	
		3	0.15	15	100	
				100	15	
		5	0.25	15	100	
				100	15	
IV	6	∞	-	15		Single sand layer
	100					
	20			15		
				100		

Table 2. Input parameters of MMC model for sand

Density	I _D (%)	ϕ_p (deg.)	ϕ_{cv} (deg.)	ψ_p (deg.)	ϕ_{ini} (deg.)	Reference
Very loose	15	32	32	0	32	Bolton [23]
Loose	35	34.25	32	4.69	32	Lee et al. [15]
Medium dense	65	38.75	32	14.06	32	Hu et al. [16]
Very Dense	100	44	32	25	32	see Figure 4

Table 3. Input parameters for comparison study

Guideline	ϕ (deg.)	N_γ	d_γ	s_γ	N_q	d_q	s_q	F_{mob}	Remarks
SNAME (2008)	27	9.32	1	0.6	13.2	#	1.45	-	Reduced ϕ by 5°
InSafeJIP (2011)	32	15*	1	-	23.2	#	1.62	0.38	Adopted F_{mob}
ISO (2012)	32	15	1	-	23.2	#	-	-	Realistic ϕ

* N_γ : a given value by Cassidy & Houlsby [30] with roughness factor 0.5

d_q : $1 + 2\tan\phi(1-\sin\phi)^2 d/D$

503 **No of Figure: 12**

- 504 **Figure 1.** Circular footing in layered sand deposits
- 505 **Figure 2.** Schematic diagram of installed spudcan in two-layered sand deposits
- 506 **Figure 3.** Typical mesh used in 3D LDFE analysis
- 507 **Figure 4.** Variation of friction and dilation angle for MMC model (after Hu et al. [16])
- 508 **Figure 5.** Comparison between LDFE results and measured data
- 509 **Figure 6.** Soil failure mechanisms for loose-over-dense sand ($I_{Dt} = 15\%$; $I_{Db} = 100\%$; in
510 Group I and Group IV, Table 1): (a) Resistance profiles; (b) $d = 0\text{m}$; (c) $d = 2.5$
511 m ; (d) $d = 5\text{ m}$; (e) $d = 8.8\text{ m}$
- 512 **Figure 7.** Soil failure mechanisms for dense-over-loose sand ($I_{Dt} = 100\%$; $I_{Db} = 15\%$; in
513 Group I and Group IV, Table 1): (a) Resistance profiles; (b) $d = 0\text{m}$; (c) $d = 2.5$
514 m ; (d) $d = 5\text{ m}$; (e) $d = 10\text{ m}$
- 515 **Figure 8.** Mobilised friction angle during penetration ($I_{Dt} = 100\%$ and $I_{Db} = 15\%$; in
516 Group I, Table 1): (a) before shoulder embedment; (b) $d = 0\text{ m}$; (c) $d = 2.5\text{m}$;
517 (d) $d = 5.0\text{ m}$ (at original interface)
- 518 **Figure 9.** Effect of relative density of sand on bearing behaviour (in Groups I and IV,
519 Table 1)
- 520 **Figure 10.** Effect of thickness of top sand layer on bearing behaviour (in Group I, Table
521 1)
- 522 **Figure 11.** Effect of spudcan size on bearing behaviour (in Groups I~IV, Table 1): (a)
523 Bearing pressure profiles; (b) Dimensionless penetration resistance profiles
- 524 **Figure 12.** Comparison results with design approaches in dense-over-loose sand (Groups
525 III and IV, Table 1)

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Figure 1. Circular footing in layered sand deposits

Figure 2. Schematic diagram of installed spudcan in two-layered sand deposits

Figure 3. Typical mesh used in 3D LDFE analysis

Figure 4. Variation of friction and dilation angle for MMC model (after Hu et al. [16])

Figure 5. Comparison between LDFF results and measured data

Figure 6. Soil failure mechanisms for loose-over-dense sand ($I_{Dt} = 15\%$; $I_{Db} = 100\%$; in Group I and Group IV, Table 1)

Figure 7. Soil failure mechanisms for dense-over-loose sand ($I_{Dt} = 100\%$; $I_{Db} = 15\%$; in Group I and Group IV, Table 1)

Figure 8. Mobilised friction angle during penetration ($I_{Dt} = 100\%$ and $I_{Db} = 15\%$; in Group I, Table 1): (a) before shoulder embedment; (b) $d = 0$ m; (c) $d = 2.5$ m; (d) $d = 5.0$ m (at original interface)

Figure 9. Effect of relative density of sand on bearing behaviour (in Groups I and IV, Table 1)

Figure 10. Effect of thickness of top sand layer on bearing behaviour (in Group I, Table

1)

(a) Bearing pressure profiles

(b) Dimensionless penetration resistance profiles

Figure 11. Effect of spudcan size on bearing behaviour (in Groups I–IV, Table 1)

Figure 12. Comparison results with design approaches in dense-over-loose sand (Groups III and IV, Table 1)