Deep-penetrating spudcan foundations on layered clays: numerical analysis

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This paper presents results of large deformation finite element (LDFE) analyses that investigate the penetration response of deep-penetrating spudcan foundations in stratified soil comprising a strong clay layer overlying weaker clay. Such strength profiles give rise to potential punch-through failure, with a local maximum penetration resistance followed by some reduction. The paper is a companion paper to one that presents results from centrifuge model tests of the same problem, with the overall aim being to investigate the potential for punch-through failure and its severity, as a basis for improving design methods for evaluating spudcan penetration. The LDFE analyses have simulated continuous penetration of smooth and rough spudcan foundations from the sea-bed surface. A detailed parametric study has been undertaken, exploring the relevant range of layer thickness (relative to the spudcan diameter), strength ratios and spudcan base roughness. The results have been validated against previously published results and the centrifuge test data presented in a companion paper. Excellent agreement was obtained between the results from LDFE analyses and centrifuge tests. As the ratio of the shear strength of the bottom layer to that of the top layer decreased, upwards soil flow around the spudcan shoulder became less pronounced and the failure mechanism was dominated by a steep-sided plug of soil from the top layer being forced deep into the lower layer. The potential for severe punch-through was demonstrated by a significant reduction in the resistance profile as the spudcan approached the interface between the two layers. A preliminary design chart is proposed to estimate the cavity depth above the penetrating spudcan, which was significantly greater than that for a single layer clay. The penetration responses have been presented in terms of profiles of normalised net bearing pressure for a range of layer geometries and soil properties. Potential punch-through occurred for all cases where the strength ratio was less than or equal to 0.6, with the reduction in resistance being greater the lower the strength ratio of the underlying layer to the upper layer, and the thicker the upper layer, but reducing as the strength gradient in the lower layer increased. The current approach suggested in offshore design guidelines is shown to underestimate the penetration resistance significantly, in most cases, and give poor estimates of the likelihood and severity of spudcan punch-through.

KEYWORDS: bearing capacity; clays; failure; footings/foundations; numerical modelling; offshore engineering

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La présente communication présente les résultats d'analyses de grandes déformations en éléments finis (LDEF) examinant la réaction à la pénétration de fondations à forte pénétration dans des sols stratifiés, comportant une couche d'argile résistante recouvrant une couche d'argile plus faible. Ces profils de résistance donnent lieu à un risque de rupture à « *punch-through* », dans le cadre de laquelle une résistance maximale locale à la pénétration est suivie d'une certaine réduction. La présente communication doit être lue conjointement avec celle qui présente les résultats de tests effectués sur maquettes centrifuges relativement au même problème, dans le but général d'examiner le potentiel de rupture par « *punch-through* », et sa sévérité, comme élément de base pour l'optimisation de méthodes d'essai permettant d'évaluer la pénétration des caissons d'enfichage. Dans le cadre des analyses LDEF, on a simulé la pénétration continue de fondations lisses et brutes des caissons d'enfichage depuis la surface du fond marin. On a effectué une étude paramétrique détaillée explorant la plage pertinente de l'épaisseur des couches (relativement au diamètre des caissons d'enfichage), les ratios de résistance, et la rugosité de la base des caissons d'enfichage. Les résultats ont été validés conformément à des résultats publiés précédemment, et les données découlant des essais centrifuges sont fournies dans la communication parallèle. On relève un accord parfait entre les résultats des analyses de LDEF et les tests centrifuges. Au fur et à mesure de la diminution du ratio de la résistance au cisaillement de la couche inférieure par rapport à la couche supérieure, la solifluxion vers le haut autour de l'épaule des caissons d'enfichage se fait moins prononcée, et le mécanisme de rupture est dominé par un bouchon de sol de la couche supérieure, aux côtés escarpés, s'enfonçant en profondeur dans la couche inférieure. Le potentiel d'un « *punch-through* » prononcé est démontré par la réduction significative du profil de résistance au fur et à mesure que le caisson d'enfichage s'approche de l'interface entre les deux couches. Un tableau conceptuel préliminaire est proposé afin d'estimer la profondeur de la cavité au-dessus du caisson d'enfichage pénétrant, qui était fort supérieure à celle d'une couche d'argile unique. Les réactions à la pénétration sont présentées sous forme de pression d'appui nette normalisée pour une série de géométries des couches et de propriétés du sol. Le « *punch-through* » potentiel se réalise chaque fois que le ratio des résistances est inférieur ou égal à 0,6, la réduction de la résistance augmentant plus le ratio de résistance de la couche sous-jacente à la couche supérieure est bas, et plus la couche supérieure est épaisse, mais baissant au fur et à mesure de l'augmentation du gradient de la résistance dans la couche inférieure. On démontre que la méthode actuelle, proposée dans les principes directeurs des études offshore, sous-estime de façon significative, dans la plupart des cas, la résistance à la pénétration, et fournit des estimations médiocres de la probabilité des « *punch-through* » des caissons d'enfichage, et de leur sévérité.

INTRODUCTION

Mobile jack-up rigs are used widely in the offshore oil and gas industry for installing small platforms, maintenance work and drilling and even for production for fields of limited life. During operations they rest on spudcan foundations, which are routinely pre-loaded by ballasting the hull of the rig and can penetrate by up to three diameters into the sea-bed. A relatively significant rate of accidents persists, particularly where the sea-bed comprises strong soil overlying weaker material (MSL, 2004; Jack *et al.*, 2007). This paper presents results of large deformation finite element (LDFE) analyses undertaken to explore the penetration response for the particular case of a strong clay layer overlying weaker clay, and to evaluate the potential for punch-through (defined by a local maximum in the penetration resistance) and its severity.

The paper is a companion to one that presents results from centrifuge model tests of the same problem (Hossain & Randolph, 2010). That paper contains extensive background information to the problem, which is not repeated here. For convenience, Fig. 1 from that paper is repeated here, showing a typical spudcan geometry and defining the nomenclature adopted for the problem.

As noted in the companion paper, current design methods are based on data from laboratory floor model tests with limited penetration of the surface footing, from which limit equilibrium design methods have been developed. More recently the problem has been addressed through finite

Fig. 1. Schematic of embedded spudcan foundation showing idealised open cavity

element analysis, although mostly limited to conventional small strain analysis of surface footings (Edwards & Potts, 2004; Merifield & Nguyen, 2006). The papers explored a range of normalised layer thickness t/D from 0.1 to 2.5, and strength ratio s_{ub}/s_{ut} from 0.14 to 1, and the latter paper also showed displacement vectors illustrating the different failure mechanisms.

The only previous numerical analysis to consider continuous penetration from the surface, using LDFE analysis, is the study by Wang & Carter (2002), who considered plane strain and circular foundations with a smooth base. Bearing capacity factors were presented graphically in terms of the thickness ratio of the top layer and the shear strength ratio encompassing $t/D = 0.5$ and 1 and $s_{ub}/s_{ut} = 0.1$ to 1. The profiles of bearing capacity factor, N_c (ratio of average bearing pressure, q , to the shear strength of the top layer, s_{ut}) calculated from the LDFE analyses, during continuous penetration, were significantly different from those from the small strain finite element analyses, resulting in higher maximum bearing capacity factors (N_{cmax}) from the former method, as shown in Fig. 2. A key finding was that the cumulative displacements around the continuously penetrating foundation play an important role in the bearing response, with a gradual thinning of the upper layer. Although large deformation finite element analyses were undertaken, so that the changing geometry with progressive penetration was correctly modelled, the footing was modelled as a cylindrical object with smooth rigid shaft and the soil weight was ignored (although this was explored for the plane strain geometry). As such, the results have limited applicability for a spudcan foundation where soil is free to flow back on top of the spudcan.

The aim of the paper is to complement the experimental work described in the companion paper (Hossain & Randolph, 2010) by means of LDFE analyses. The analytical study considers spudcan penetration from the surface, through an overlying layer of uniform strength clay with self-weight, into a softer underlying layer with either homogeneous shear strength or a linearly increasing shear strength. Both fully smooth and fully rough interfaces between the spudcan and the clay are investigated. Details of the analysis method are summarised briefly and the results are then validated through comparisons of the deformation fields and profiles of bearing resistance with results from the

Fig. 2. Bearing capacity factors of circular foundations presented in literature

centrifuge model tests. An expression is developed to estimate the maximum depth of cavity that forms above the spudcan before back-flow of soil occurs, comparing the results with previous work in a single layer clay (Hossain *et al.*, 2005). Profiles of bearing resistance for use in design are presented for a range of layer geometries, t/D , strength ratios, $s_{\text{ubs}}/s_{\text{ut}}$, normalised strengths, $s_{\text{ubs}}/\gamma'_b D$, and for different rates of strength increase of the bottom layer, kD/s_{ubs} (see Fig. 1). It is hoped that the numerical results, combined with the visual evidence from both physical and numerical modelling, will prompt a reappraisal of the conventional design approach and point the way forwards for the development of a more rational design approach based on the actual failure mechanisms.

NUMERICAL ANALYSIS

Geometry and parameters

This study has considered a circular spudcan foundation of diameter D , penetrating into a two-layer deposit as illustrated schematically in Fig. 1, where the top layer of clay with uniform undrained shear strength s_{ut} , effective unit weight γ'_t , and thickness t is underlain by a clay layer of non-uniform undrained shear strength $s_{\text{ub}} = s_{\text{ubs}} + k(z-t)$, effective unit weight γ'_b , and (nominally) infinite depth. Analyses were undertaken for spudcans with a 13° shallow conical underside profile (included angle of 154°) and a 76° protruding spigot of height $0.14D$ (see flow mechanism figures such as Fig. 3, also Fig. 4 of Hossain & Randolph, 2009a). The shape was chosen to mimic the centrifuge spudcan model, which was fabricated based on the spudcans used in practice with the jack-up rig 'Marathon LeTourneau Design, Class 82-SDC', as illustrated by Menzies & Roper (2008). The soil–spudcan interface was modelled as either fully smooth or fully rough, using nodal joint elements (Herrmann, 1978).

Practical ranges of soil properties, as reported in the literature, are summarised by Hossain & Randolph (2009a). The maximum top layer undrained strength of practical interest is taken as $s_{\text{ut}} = 200$ kPa, with minimum bottom layer strength of $s_{\text{ubs}} = 10$ kPa, and lower layer strength gradient k typically varying between 0 and 2 kPa/m. The effective unit weight of the bottom layer is typically $\gamma'_b = 6.5$ to 7.5 kN/m³ and that of the top layer is $\gamma'_t = 7.5$ to 8.5 kN/m³. The effective (equivalent area) diameter of modern spudcans ranges from 10 to 20 m.

In this study, the values of s_{ub} and spudcan diameter were selected so that normalised lower layer strength at the interface varied as $s_{\text{ubs}}/\gamma'_b D = 0.12, 0.36, 0.60$, increasing with depth with degree of non-homogeneity (kD/s_{ubs}) of 0 to 2, while values of s_{ut} were determined according to strength ratios $s_{\text{ubs}}/s_{\text{ut}} = 0.2, 0.4, 0.6$ and 0.8 . The submerged unit weight for the stiff and soft clay layers were chosen with a ratio $\gamma'_b/\gamma'_t = 0.87$. The thickness of the top stiff layer was varied relative to the spudcan diameter as $t/D = 0.5, 1.0, 1.5, 2.0$. The selected parameters for this study are assembled in Table 1, encompassing most cases of practical interest.

Analysis details

Finite element analyses were performed using the AFENA finite element package (Carter & Balaam, 1990) developed at the University of Sydney. H -adaptive mesh refinement cycles (Hu & Randolph, 1998b) were implemented to optimise the mesh, minimising discretisation errors, concentrating in the most highly stressed zones. Large deformation analyses were undertaken using RITSS (remeshing and interpolation technique with small strain; Hu & Randolph,

Fig. 3. Comparison of soil flow mechanisms between centrifuge test and LDFE analysis ($s_{\text{ubs}}/s_{\text{ut}} = 0.62$, $s_{\text{ubs}}/\gamma'_b D = 0.28$, $t/D = 1$; group I, Table 1): (a) $d/D = 0.38$, (b) $d/D = 1.15$

Fig. 4. Comparison of load-penetration response between centrifuge test and LDFE analysis ($s_{\text{ubs}}/s_{\text{ut}} = 0.62$, $s_{\text{ubs}}/\gamma'_b D = 0.28$, $t/D = 1$; group I, Table 1)

Table 1. Summary of LDFE analyses performed

Analysis	$s_{\text{ubs}}/\gamma'_b D$	$s_{\text{ubs}}/s_{\text{ut}}$	kD/s_{ubs}	t/D	γ'_b/γ'_t	Spudcan base	Notes
Group I	0.28	0.62	0	1	0.97	Rough and smooth	Comparison with centrifuge test results
Group II		0.1, 0.2, 0.5, 0.67, 0.8 and 1	0	1	Weightless	Smooth	Comparison with Wang & Carter's (2002) results
Group III	0.12	0.1, 0.2, 0.5, 0.67, 0.8 and 1	0	1	0.87	Smooth	Investigation of the effect of soil self-weight
Group IV	0.12	0.2, 0.4, 0.6, 0.8 and 1	0	0, 0.5, 1, 1.5, 2 and ∞	0.87	Rough and smooth	Parametric study for uniform-over-uniform clays
Group V	0.36	0.2, 0.4, 0.6 and 0.8	0	0.5, 1, 1.5 and 2	0.87	Rough and smooth	
Group VI	0.60	0.2, 0.4, 0.6 and 0.8	0	0.5, 1, 1.5 and 2	0.87	Rough and smooth	
Group VII	0.36	0.2, 0.4, 0.6 and 0.8	0.4, 0.5, 0.6, 0.67, 0.7, 0.8, 1 and 2	0.5, 1, 1.5 and 2	0.87	Rough and smooth	Exploration of the effect of lower layer strength non-homogeneity

1998a). This method falls within what are known as arbitrary Lagrangian–Eulerian (ALE) finite element methods (Ghosh & Kikuchi, 1991), whereby a series of small strain analysis increments (using AFENA) are combined with fully automatic remeshing of the entire domain, followed by interpolation of all field values (such as stresses and material properties) from the old mesh to the new mesh. Other applications of this technique include circular footing penetration in two-layer clay (Wang & Carter, 2002), cone penetration in clay (Lu *et al.*, 2004) and uplift of strip anchors in clay (Thorne *et al.*, 2004).

Details of the LDFE approach for analysing spudcan penetration have been given by Hossain *et al.* (2005). Penetration of the spudcan is simulated by specifying incremental displacements. The displacement increment size and the number of steps of small strain analysis between each re-meshing were chosen such that the cumulative penetration between re-meshing stages remained in the small strain range and was less than half the minimum element size, as suggested by Hu *et al.* (1999). Six-noded triangular elements with three internal Gauss points were used in all the FE analyses.

The axisymmetric soil domain was chosen as $12D$ in diameter and $10D$ in depth to ensure that the boundaries were well outside the plastic zone. The soil was modelled as an elasto-plastic material obeying a Tresca yield criterion, with associated flow apart from rounding of the corners of the hexagon (Sloan & Booker, 1986). 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), friction and dilation angles $\phi = \psi = 0$, and a uniform stiffness ratio $E/s_u = 500$ (where E is the Young's modulus) throughout the soil profile. The stiffness ratio is within the range commonly adopted for soft clays, but the precise value has negligible effect on the results presented. Typical computation times on a fast desktop computer were about 24 h for a spudcan penetration of two diameters. In the response curves presented later, the spudcan penetration is plotted relative to the depth of the lowest maximum diameter of the spudcan below the original ground surface.

BEHAVIOUR OF SPUDCAN FOUNDATIONS

Comparison between LDFE results and centrifuge test data

A comparison is presented here between one of the model tests (test E1UU-III-T 7, Table 3; Hossain & Randolph,

2009a) and an LDFE simulation. The test modelled a spudcan of prototype diameter 6 m penetrating a (nominally) uniform strength top layer, with $t/D = 1$, overlying a uniform strength bottom layer. The actual undrained shear strength profile, assessed in-flight from a T-bar penetrometer test, was idealised for the top layer as $s_{\text{ut}} = 6.5 + 7.5z$ kPa for depth $z < 1.6$ m and $s_{\text{ut}} = 17.86 + 0.4z$ kPa for $z > 1.6$ m, and for the bottom layer as $s_{\text{ub}} = s_{\text{ubs}} = 12.3$ kPa. The average clay effective unit weights were 7.5 and 7.3 kN/m³ for the upper and lower layers respectively.

Figure 3 shows the soil flow mechanisms from LDFE analysis using the above parameters and adopting a rough-based spudcan (group I, Table 1; note, the relative roughness of the spudcan base appears to have little influence as will be discussed later). The displacement vectors obtained from the centrifuge test using particle image velocimetry (PIV) analyses are shown for comparison on the left. It can be seen that at shallow penetration (Fig. 3(a)), the soil at the spudcan edge flows upwards to the surface and the soil beneath the central part of the spudcan is forced downwards to the lower layer, forming a cavity above the spudcan and deforming the layer interface, and with less surface heave compared to the case of a single layer clay (Hossain *et al.*, 2005). The cavity remains completely open through the top layer and back-flow of soil around the spudcan into the cavity is initiated at a penetration depth of about $d/D = 1.08$. Fig. 3(b) shows the flow pattern just after that critical depth. A bowl shaped strong soil plug is carried down beneath the spudcan into the soft layer, while material from the stronger upper layer has started to flow back over the spudcan top. No clear shear surfaces are evident in the upper layer (for example, in Fig. 3(a)). Instead, failure has occurred through a combination of general shear (at the edge of the spudcan) and partial punching shear, or cavity expansion mode, for this relatively higher strength ratio (0.62).

Excellent agreement can be seen between the results from the centrifuge model test and LDFE analysis in terms of key features of the flow mechanisms: surface heave, cavity depth, layer deformation, trapped stronger material beneath the spudcan and the failure pattern generated owing to penetration of the spudcan foundation. The corresponding load–penetration responses (from the corresponding full-spudcan centrifuge test) are also consistent, as shown in Fig. 4, particularly after minor adjustment to allow for effects of partial remoulding of the soil as the spudcan penetrates (Hossain & Randolph, 2009a).

The good agreement with the model test provides confidence in the accuracy of the LDFE results. The results of the parametric study covering a wide range of layer soil properties and geometry, as assembled in Table 1, are now discussed.

Punch-through failure mechanism

Figure 5 depicts the soil flow mechanisms during penetration of a spudcan in a soil with very low strength ratio of $s_{\text{ubs}}/s_{\text{ut}} = 0.2$ ($s_{\text{ubs}}/\gamma'_b D = 0.36$, $t/D = 1$; in group V, Table 1), showing the mechanism for the most severe punch-through failure obtained in the study. The flow patterns revealed at different stages of spudcan penetration are also entirely consistent with the centrifuge observations (see Fig. 7 of Hossain & Randolph, 2009a). From the start of penetration the soil deformation is directed predominantly vertically

downwards to the lower layer, with no upwards movement (see Fig. 5(a)). A significant punch-through, or reduction in bearing capacity, occurs at $d/D = 0.2$ (see the corresponding resistance profile later in Fig. 10(b)), at which stage a shear surface bounds a truncated cone of material in the upper layer, which is forced into the softer underlying layer. With further penetration ($d/D = 0.57$; Fig. 5(a)), the vertical flow remains confined laterally directly beneath the spudcan, with plastic yielding extending deep into the lower layer. New approximately vertical shear planes (part of a cylindrical shear surface) are observed along with a transitional zone of shear deformation comprising downwards and outwards motion.

When the spudcan base is at the original level of the layer interface ($d/D = 1$; Fig. 5(b)), an inverted truncated cone of material from the upper layer has been forced down into the underlying soft layer soil, with deformation extending deep into the lower layer, but with increasing outwards flow. The initial layer interface, and to a lesser extent the original soil surface, are raised slightly to accommodate the foundation with the trapped plug (although this may be partly attributable to the conditions enforced in the analysis, with fixed base and outer boundaries). The stronger soil within the transitional shear deformation zone has not started to flow back over the spudcan and the cavity remains fully open.

Figure 5(b) also shows the flow pattern at $d/D = 1.25$ after the spudcan had penetrated into the lower layer. Stronger material around the spudcan edges is gradually tending to flow back into the cavity above the spudcan. The soil plug with a hemispherical bottom is now entirely within the soft layer, with the depth of the trapped strong soil plug about 75% of the original thickness of the strong layer.

The mechanisms in Figs 3 and 5 illustrate the effect of the relative strength of the bottom layer; the weaker the layer, the more the failure is confined within the lower layer (together with distinct shear planes through the upper layer) and the more severe is the eventual punch-through. Similar flow patterns were found for the smooth-based spudcan, and hence are not reiterated here, although a slightly shorter plug of strong soil was trapped beneath the spudcan and forced down into the lower layer.

Fig. 5. Critical punch-through mechanism for spudcan penetration in stiff-over-soft clay ($s_{\text{ubs}}/s_{\text{ut}} = 0.2$, $s_{\text{ubs}}/\gamma'_b D = 0.36$, $t/D = 1$; in group V, Table 1): (a) $d/D < t/D$, (b) $d/D \geq t/D$

Depth of soil back-flow: limiting cavity depth

From observations of the soil flow mechanisms associated with continuous penetration of a spudcan foundation, a stage of penetration can be identified where the soil starts to flow back into the cavity above the spudcan. This is referred to as the depth of back-flow, and is a function of the layer parameters and geometry ($s_{\text{ubs}}/s_{\text{ut}}$, $s_{\text{ubs}}/\gamma'_b D$, t/D , kD/s_{ubs}). The subsequent back-flow provides a seal above the spudcan and limits the cavity depth.

By studying the flow mechanisms during continuous penetration, the depth of back-flow initiation and the average cavity depth at the end of the penetration were identified from all the analyses tabulated in Table 1. It was found that the depth of back-flow was approximately equal to the (average) limiting cavity depth, H .

All the results are presented in Fig. 6 with the normalised cavity depth, H/D , plotted against $s_{\text{uH}}/\gamma'_b D$, where s_{uH} and γ'_b are the shear strength and effective soil density at the back-flow depth, H (equal to s_{ut} and γ'_t for $H < t$ and s_{ub} and γ'_b for $H > t$). Although spudcan base roughness was found to have little influence on the critical depth, the values for both cases are presented together. The relationship developed previously for uniform soil ($s_{\text{ut}} = s_{\text{ub}} = s_{\text{uH}}$) (Hossain *et al.*, 2005)

Fig. 6. Cavity depth, H , during spudcan installation in stiff-over-soft clay

$$\frac{H}{D} = \left(\frac{s_{uH}}{\gamma'_H D} \right)^{0.55} \quad (1)$$

has also been included in the figure. Values for the two-layered case where the cavity is confined to the upper layer ($H/D < t/D$: open symbols) lie close to the design line proposed for a single layer clay. However, where the cavity depth exceeds the layer thickness ($H/D > t/D$: filled symbols), the values are significantly higher, increasing as the strength ratio reduces for any thickness ratio. The presence of an underlying soft layer allows the soil beneath the spudcan to be pushed downwards, rather than back-flow occurring in the stronger material surrounding the spudcan, thus delaying initiation of back-flow.

All cavity depths obtained from the LDFE analyses and centrifuge tests (for $H/D > t/D$) are plotted in Fig. 7, as a function of normalised soil parameters and layer geometry, $(s_{ut}/\gamma'_b D)(t/D)/(1 + kD/s_{ubs})$. The values measured in the centrifuge tests are in good agreement with the LDFE results, and interestingly all the data show a unique trend, which may be expressed as

$$\frac{H}{D} \approx 1.4 \left\{ \left(\frac{s_{ut}}{\gamma'_b D} \right) \left(\frac{t}{D} \right) / \left(1 + \frac{kD}{s_{ubs}} \right) \right\}^{0.5} \quad (2)$$

In principle, this relationship should only be used where it yields a depth of back-flow greater than the layer thickness ($H/D > t/D$). However, where the calculated depth is less than the layer thickness, the actual depth of back-flow appears to lie between estimates from equations (1) and (2), with a gradual transition towards the former as H/t decreases.

Fig. 7. Design chart for estimating cavity depth after spudcan installation in stiff-over-soft clay

Bearing response: general behaviour

The load-penetration responses are presented in terms of normalised net bearing pressure, q_{net}/s_{ubs} , as a function of normalised penetration depth, d/D , with q_{net} calculated as

$$q_{net} = \frac{P}{A} - \gamma'_t \frac{V}{A} \quad (3)$$

where P is the penetration resistance, V is the volume of the embedded spudcan including shaft and A is the largest cross-sectional area of the spudcan. The bearing response of spudcan foundations on layered clays is affected by a num-

ber of factors (as is also evident in the soil flow mechanisms)

- (a) the strength ratio between lower and upper soil layers, $s_{\text{ubs}}/s_{\text{ut}}$
- (b) the thickness of the upper layer relative to the spudcan diameter, t/D
- (c) the normalised clay strength, $s_{\text{ubs}}/\gamma'_b D$
- (d) the strength gradient of the lower layer, denoted by a 'non-homogeneity factor', kD/s_{ubs}
- (e) the ratio of soil unit weights, γ'_b/γ'_i
- (f) the roughness of the spudcan base.

The effects of the different ratios emerged to some extent from the centrifuge tests (Hossain & Randolph, 2010), but are now evaluated from the more extensive LDFE parametric study, as summarised in Table 1. Note that in practice the ratio of soil unit weights lies around 0.8 to 0.9, and here a constant ratio of 0.87 has been adopted for the main study.

Analyses without soil weight. Initially, analyses were performed for weightless soil ($\gamma'_i = \gamma'_b = 0$) in order to confirm the accuracy of the capacities computed compared with the work of Wang & Carter (2002). Fig. 8 plots the normalised net bearing pressure, $q_{\text{net}0}/s_{\text{ubs}}$ (where the subscript 0 denotes weightless soil) for a smooth-based spudcan penetrating into layered soils with $t/D = 1$ and $s_{\text{ubs}}/s_{\text{ut}} = 0.1, 0.2, 0.5, 0.67, 0.8$ and 1 (group II, Table 1). The figure also includes results from LDFE analyses for a circular flat footing (Wang & Carter, 2002). The cavity above the advancing spudcan remained open up to full installation in these weightless soils, mimicking the situation for a cylindrical foundation with smooth vertical sides. The base apex angles are relatively similar (154° for the spudcan and 180° for the circular flat footing), so that similar responses should be obtained. Indeed, both sets of results are in excellent agreement with variations mostly less than 5%, confirming the accuracy of the results of the present analyses. Although for the layered soil cases the analyses have not been taken to sufficiently deep penetration into the bottom layer to reach a steady state, there is a clear indication that the bearing curves are tending to approach that for the single layer case ($s_{\text{ubs}}/s_{\text{ut}} = 1$).

To examine the effect of soil self-weight (normalised as $s_{\text{ubs}}/\gamma'_b D$) on the spudcan bearing response, further analyses were performed for the above cases assuming a unit weight ratio of $\gamma'_b/\gamma'_i = 0.87$ and normalised shear strength of $s_{\text{ubs}}/\gamma'_b D = 0.12$ (group III, Table 1). The normalised bearing pressures are plotted in Fig. 9. The results shown in Fig. 8 may be used to estimate the response for soil with self-weight by including the contribution of surcharge pressure (at least while the cavity remains fully open), according to

$$q_{\text{net}} = q_{\text{net}0} + \gamma'_i d \text{ for } d \leq t \quad (4a)$$

and

$$q_{\text{net}} = q_{\text{net}0} + \gamma'_i t + \gamma'_b (d - t) \text{ for } d > t \quad (4b)$$

These are shown as grey lines in Fig. 9. It can be seen that superposition of the contributions from the soil strength and the overburden stress gives adequate predictions of the net bearing pressure up to the critical depth of soil back-flow above the spudcan, as expected. For penetrations beyond the maximum cavity depth ($d > H$), the results based on equations (4a) and (4b) gradually diverge from the true response, over-estimating the bearing pressure. This discrepancy may be attributed to the presence of backfilled soil above the spudcan, which (partially) negates the increasing surcharge contribution through the additional pressure applied above the spudcan. Therefore, it can be concluded that the results for weightless soil may be used in practice to

Fig. 8. Bearing pressure for smooth-based circular foundations in weightless soil ($t/D = 1$; group II, Table 1)

Fig. 9. Comparison between the results for soil with self-weight and predictions from equations (4a) and (4b) ($s_{\text{ubs}}/\gamma'_b D = 0.12$, $t/D = 1$, smooth; group III, Table 1)

estimate the bearing response making use of equations (3) and (4), but only while a cavity persists above the spudcan, with the maximum cavity depth estimated using equation (2).

Effect of strength ratio ($s_{\text{ubs}}/s_{\text{ut}}$). In order to explore the effect of the strength ratio on the form of the penetration resistance profile, the results of various strength ratios $s_{\text{ubs}}/s_{\text{ut}} = 0.2, 0.4, 0.6$ and 0.8 , with $s_{\text{ubs}}/\gamma'_b D = 0.12, 0.36$ and 0.60 , are plotted in Fig. 10 for a constant thickness ratio of $t/D = 1$ (in groups IV to VI, Table 1). The potential for punch-through failure exists for all strength ratios for soil with low normalised strength ($s_{\text{ubs}}/\gamma'_b D$), and for $s_{\text{ubs}}/s_{\text{ut}} \leq 0.6$ for the two higher normalised strengths, with the severity becoming greater for lower strength ratios. The increasing punch-through severity with lower strength ratios is caused by a change in the corresponding soil flow mechanisms from a combination of general shear and partial punching shear

Fig. 10. Effect of strength ratio ($s_{\text{ubs}}/s_{\text{ut}}$) on bearing response ($t/D=1$, Table 1): (a) $s_{\text{ubs}}/\gamma'_b D=0.12$ (rough; in group IV), (b) $s_{\text{ubs}}/\gamma'_b D=0.36$ (in group V), (c) $s_{\text{ubs}}/\gamma'_b D=0.60$ (rough; in group VI)

failure to full punching shear failure, as demonstrated in Figs 3 and 5. Consequently, for high normalised strengths and relatively higher strength ratios, the resistance profile rises monotonically at least up to some penetration depth before a gradual punch-through occurs (e.g. $s_{\text{ubs}}/s_{\text{ut}}=0.6$ and $s_{\text{ubs}}/\gamma'_b D > 0.12$; Figs 10(b) and (c)). However, for low strength ratio of $s_{\text{ubs}}/s_{\text{ut}}=0.2$, a significant reduction in bearing capacity, that is severe punch-through, occurs at shallow penetration (see Figs 10(b) and (c)). This full punch-through involves the formation of a distinct cylindrical shear surface through the upper layer, and primarily downwards soil deformation extending deep into the lower layer (see Fig. 5).

For this thickness ratio ($t/D=1$), for cases of high strength ratio and low normalised strength (e.g. $s_{\text{ubs}}/s_{\text{ut}}=0.8$ and 0.6 for $s_{\text{ubs}}/\gamma'_b D=0.12$; Fig. 10(a)), soil flows back into the cavity above the spudcan well within the top layer. The bearing response shows a plateau before gradual reduction, with the maximum resistance occurring just before the spudcan reaches the lower layer. The cases where no punch-through occurs (high strength ratio and high normalised strengths) show the bearing resistance increasing monotonically up to a plateau (although note that if the shear strength profile in the lower layer is indeed uniform, the constant penetration resistance occurring for $d/D > 1$ could still lead to a toppling instability).

For cases where the cavity depth exceeds the thickness of the upper layer, once back-flow is initiated, a more obvious reduction of bearing pressure occurs even where punch-through has already taken place (e.g. $s_{\text{ubs}}/s_{\text{ut}}=0.4$ for $s_{\text{ubs}}/\gamma'_b D=0.36$; Fig. 10(b)) and also a punch-through is triggered for monotonically rising profiles (e.g. $s_{\text{ubs}}/s_{\text{ut}}=0.4$ for $s_{\text{ubs}}/\gamma'_b D=0.12$; Fig. 10(a)). The effect of strength ratio discussed above is consistent with observations from the centrifuge tests.

Effect of thickness ratio (t/D). In order to demonstrate the effect of the relative thickness of the upper layer, bearing resistance profiles are plotted in Figs 11(a) to (c) for a range of t/D from 0.5 to 2, but with identical strength ratios respectively of $s_{\text{ubs}}/s_{\text{ut}}=0.6$ ($s_{\text{ubs}}/\gamma'_b D=0.12$; in group IV, Table 1), $s_{\text{ubs}}/s_{\text{ut}}=0.4$ ($s_{\text{ubs}}/\gamma'_b D=0.36$; in group V, Table 1) and $s_{\text{ubs}}/s_{\text{ut}}=0.2$ ($s_{\text{ubs}}/\gamma'_b D=0.60$; in group VI, Table 1). The penetration resistance profiles for each strength ratio and corresponding normalised strength show a consistent trend as the thickness ratio, t/D , increases.

The responses on layered deposits are bracketed by those for a single layer clay with a shear strength equal either to that in the top (s_{ut}) or bottom (s_{ub}) layer, as shown in Fig. 11(a). In the upper layer, the difference in bearing capacity values for the layered deposits compared to the response for a single layer clay with shear strength s_{ut} increases as the crust thickness ratio decreases. For $t/D=0.5$, the curve departs almost immediately from that corresponding to the upper soil conditions, owing to the influence of the soft layer. For thicker crusts with $t/D \geq 1$, the curves follow the response for the upper soil layer to a certain penetration before diverging as the soft layer is detected. The proximity to the layer interface before the curves diverge decreases with decreasing strength ratio. For instance, for $s_{\text{ubs}}/s_{\text{ut}}=0.6$ ($s_{\text{ubs}}/\gamma'_b D=0.12$, Fig. 11(a)), the curves diverge at approximately $0.5D$ above the interface, but that distance has increased to at least $1D$ for $s_{\text{ubs}}/s_{\text{ut}}=0.4$ ($s_{\text{ubs}}/\gamma'_b D=0.36$, Fig. 11(b)). This trend is also exacerbated by the increasing normalised shear strength, $s_{\text{ubs}}/\gamma'_b D$, in Figs 11(a), (b) and (c).

Figure 11 also includes another set of illustrations for high strength ratio and high normalised strength ($s_{\text{ubs}}/s_{\text{ut}}=0.8$, $s_{\text{ubs}}/\gamma'_b D=0.36$; in group V, Table 1), where no

Fig. 11. Effect of thickness ratio (t/D) on bearing response (Table 1): (a) $s_{\text{ubs}}/s_{\text{ut}} = 0.6$ ($s_{\text{ubs}}/\gamma'_b D = 0.12$, rough; in group IV), (b) $s_{\text{ubs}}/s_{\text{ut}} = 0.4$ ($s_{\text{ubs}}/\gamma'_b D = 0.36$, rough; in group V), (c) $s_{\text{ubs}}/s_{\text{ut}} = 0.2$ ($s_{\text{ubs}}/\gamma'_b D = 0.60$; in group VI), (d) $s_{\text{ubs}}/s_{\text{ut}} = 0.8$ ($s_{\text{ubs}}/\gamma'_b D = 0.36$, rough; in group V)

reduction in bearing capacity was evident for a thickness ratio of unity, as discussed previously. However, Fig. 11(d) shows that punch-through may still be triggered within the upper layer for thicker crust layers of $t/D > 1$, as the spudcan is about to penetrate into the lower layer; soil back-flow occurs well within the top layer for such cases.

At large penetrations, the bearing resistance curves for all cases tend to approach the steady ultimate resistance corresponding to the bottom layer, although the approach is quite gradual (particularly for low strength ratios), because of the plug of stronger soil carried down with the spudcan.

Interestingly, the degree of bearing capacity reduction with penetration, a measure of the severity of punch-through, increases with the relative thickness of the upper layer. The depth of the peak in penetration resistance, prior to punch-through, also increases with crust thickness ratio, as indicated by the dashed line in Fig. 11(c). In assessing spudcan response after a punch-through, and for safe re-installation on the same location, Brennan *et al.* (2006) came to a

similar conclusion that the potential for severe punch-through decreased with decreasing thickness of the stronger layer. These effects are also similar to those found in the centrifuge tests.

Effect of normalised strength ($s_{\text{ubs}}/\gamma'_b D$). The effect of the normalised shear strength ($s_{\text{ubs}}/\gamma'_b D$) on the bearing response is evident from Fig. 10, for example comparing responses for a given strength ratio in Figs 10(a) and (b). Fig. 12 focuses specifically on this issue with the bearing responses for $s_{\text{ubs}}/\gamma'_b D = 0.12$ to 0.60 are compared directly for four different strength ratios (all for $t/D = 1$; in groups IV to VI, Table 1). When the strength ratio $s_{\text{ubs}}/s_{\text{ut}} \geq 0.6$ (Figs 12(a) and (b)), the profiles rise monotonically at least up to the layer interface except for the lowest normalised strength of $s_{\text{ubs}}/\gamma'_b D = 0.12$; the potential for punch-through failure is therefore mild. For those cases, any reduction in bearing resistance is delayed until after soil back-flow has been

Fig. 12. Effect of normalised strength ($s_{\text{ubs}}/\gamma'_b D$) on bearing response ($t/D = 1$, rough; in groups IV to VI, Table 1): (a) $s_{\text{ubs}}/s_{\text{ut}} = 0.8$, (b) $s_{\text{ubs}}/s_{\text{ut}} = 0.6$, (c) $s_{\text{ubs}}/s_{\text{ut}} = 0.4$, (d) $s_{\text{ubs}}/s_{\text{ut}} = 0.2$

initiated in the top layer. However, the reverse trend is true for lower strength ratios with $s_{\text{ubs}}/s_{\text{ut}} < 0.6$ as is clear from Figs 12(c) and (d). The maximum bearing resistance tends to occur at relatively shallow depths (except for the lowest normalised strength), and the degree of reduction in resistance increases as the normalised strength increases.

Effect of strength non-homogeneity (kD/s_{ubs}). In the above sections, the lower layer has been considered as uniform with clay strength gradient $k = 0$ ($s_{\text{ub}} = s_{\text{ubs}}$). From the centrifuge test results of Hossain & Randolph (2010), it was found that positive strength gradients in the lower layer (with $s_{\text{ub}} = s_{\text{ubs}} + k(z-t)$) have a measurable influence on the spudcan bearing behaviour. To explore this influence of strength non-homogeneity kD/s_{ubs} comprehensively, analyses have been undertaken for a clay with $s_{\text{ubs}}/\gamma'_b D = 0.36$ but varying kD/s_{ubs} between 0.4 and 2, including normally consolidated profiles where the strength profile in the lower layer is $s_{\text{ub}} = kz$, for thickness ratio of $t/D = 0.5$ to 2 (see

group VII, Table 1). Four values of $s_{\text{ubs}}/s_{\text{ut}} = 0.2, 0.4, 0.6$ and 0.8 were examined for each combination of t/D and kD/s_{ubs} .

Figures 13(a) to (d) show the bearing resistances on layered deposits with thickness ratio $t/D = 0.5, 1$ and 2 and strength ratio $s_{\text{ubs}}/s_{\text{ut}} = 0.2$ and 0.4 . This range of parameters was chosen deliberately to illustrate the effect of the lower layer strength gradient on punch-through potential. In all other cases with higher strength ratios of $s_{\text{ubs}}/s_{\text{ut}} \geq 0.6$, bearing resistance profile was found to rise monotonically, with no reduction, regardless of thickness ratio t/D and kD/s_{ubs} (≥ 0.4).

It may be seen from Fig. 13 that the strength non-homogeneity has greatest effect on the bearing capacity when t/D is small, since the failure mechanism then extends deeper into the lower layer. Consequently, the curves diverge early from that for $kD/s_{\text{ubs}} = 0$, as the influence of the increasing lower layer strength occurs almost from the start of penetration (Figs 13(a) and (c)). For thicker crusts, depending on the strength ratio, the curves follow the response for $kD/s_{\text{ubs}} = 0$ to a greater penetration prior to

Fig. 13. Effect of non-homogeneity factor (kD/s_{ubs}) on bearing response ($s_{\text{ubs}}/\gamma'_b D = 0.36$, rough; group VII, Table 1): (a) $s_{\text{ubs}}/s_{\text{ut}} = 0.4$ ($t/D = 0.5$), (b) $s_{\text{ubs}}/s_{\text{ut}} = 0.4$ ($t/D = 1$), (c) $s_{\text{ubs}}/s_{\text{ut}} = 0.2$ ($t/D = 0.5$), (d) $s_{\text{ubs}}/s_{\text{ut}} = 0.2$ ($t/D = 2$)

diverging once the effect of increasing strength is detected (Figs 13(b) and (d)). The bearing resistance profiles for $kD/s_{\text{ubs}} \geq 0.4$ also diverge from each other to an extent that depends on t/D and $s_{\text{ubs}}/s_{\text{ut}}$.

The bearing capacity for $kD/s_{\text{ubs}} \geq 0.4$ is always greater than that for $kD/s_{\text{ubs}} = 0$ owing to the greater 'average' strength of the lower layer. The likelihood of punch-through is exhibited only for $s_{\text{ubs}}/s_{\text{ut}} \leq 0.4$. Even so, the severity of punch-through reduces substantially as kD/s_{ubs} increases. It is evident from Fig. 13 that while severe punch-through happens for $kD/s_{\text{ubs}} = 0$, very mild or even no reduction in bearing resistance may occur as kD/s_{ubs} approaches unity. The explanation for the effect of kD/s_{ubs} is indicated in Fig. 13(b) by the entirely different soil flow patterns for cases where all other parameters are identical but with different non-homogeneity factors (kD/s_{ubs}) of 0 and 0.6.

Effect of spudcan base smoothness. The results of all penetration analyses presented so far were obtained considering a fully rough spudcan base. Analyses have also been

performed considering a fully smooth soil–spudcan interface to examine the effect of spudcan base smoothness on the resulting penetration resistance. In a single layer clay with strength increasing with depth, layers of softer material were always present around the advancing spudcan, which reduced the effect of base roughness and resulted in similar responses for rough and smooth-based spudcans (Hossain *et al.*, 2006). By contrast, penetration into stiff-over-soft clay generates the reverse situation, with stronger material trapped beneath the advancing spudcan. The trapped material, and delay in localised flow of soil around the spudcan, leads to little relative shearing between spudcan and soil. The shape and size of trapped material were very similar for both smooth and rough cases. Consequently, in all analyses performed, the effect of spudcan base smoothness on the bearing response was found to be minimal. Figs 4, 10(b) and 11(c) show bearing resistance profiles for smooth-based spudcans, as examples, along with corresponding profiles for a rough base. Only small reductions in the bearing pressure ($< 5\%$) were computed throughout full penetration. Similar findings with negligible effect on the calculated bearing capacity were reported by Griffiths

(1982), Merifield *et al.* (1999) and Merifield & Nguyen (2006).

Comparison with SNAME approach

Current offshore design guidelines for assessing spudcan penetration response (Society of Naval Architects and Marine Engineers (SNAME, 2002)) recommend a bearing capacity factor calculated following Brown & Meyerhof (1969), but adjusted for embedment depth by applying a constant depth factor, following the semi-empirical approach of Skempton (1951). However, Brown & Meyerhof's approach was developed on the basis of model tests conducted on a surface circular footing with very small displacement prior to failure, and under $1g$ conditions without maintaining stress similitude in terms of $s_u/\gamma'd$. Therefore their approach does not account for the distortion of the upper layer as it punches through into the lower layer.

The profile of bearing resistance estimated using the SNAME approach provides unduly conservative estimates of the penetration resistance regardless of the normalised soil properties and layer geometry, as demonstrated in the companion paper (Hossain & Randolph, 2010) and more obviously in Fig. 14. This indicates an urgent need for reassessment of the SNAME approach and evolving a more rational approach. Combining all results from this study, a new mechanism-based design approach is presented in Hossain & Randolph (2009b).

CONCLUSIONS

This paper has reported results of LDFE analyses investigating the potential for punch-through of spudcan foundations during deep penetration through a stronger clay layer into weaker material. The LDFE analyses were undertaken with the spudcan foundation penetrated continuously from the sea-bed surface. The resulting soil flow mechanisms and profiles of penetration resistance were validated against corresponding results from centrifuge model tests reported in Hossain & Randolph (2010) and other relevant published data from LDFE analyses. The effects of a wide range of normalised soil properties and layer geometry on the soil flow mechanisms, cavity depth and bearing resistance profiles were investigated, with specific focus on the likelihood of spudcan punch-through and its severity.

Overall, excellent agreement was obtained between the results from LDFE analyses and centrifuge tests in terms of soil flow features and load–penetration responses. The bearing capacity values for weightless soil were also shown to be consistent with results from Wang & Carter (2002).

The penetration resistance of spudcan foundations in stiff-over-soft clay is a complex function of a number of dimensionless factors

- the strength ratio between lower and upper soil layers, s_{ubs}/s_{ut}
- the thickness of the upper layer relative to the spudcan diameter, t/D
- the normalised clay strength, $s_{ubs}/\gamma'_b D$
- the strength gradient of the lower layer, denoted by a 'non-homogeneity factor', kD/s_{ubs}
- the ratio of soil unit weights, γ'_b/γ'_t
- the relative roughness of the spudcan base.

The parametric study covered a practical range of these parameters.

The soil self-weight, quantified in terms of the normalised shear strength, $s_{ubs}/\gamma'_b D$, plays a significant role on the evolution of soil flow patterns with spudcan penetration, and in particular on the transition depth for soil back-flow into

Fig. 14. Comparison between LDFE results and SNAME approach ($s_{ubs}/\gamma'_b D = 0.36$, $kD/s_{ubs} = 0$, rough; in group V, Table 1): (a) $s_{ubs}/s_{ut} = 0.2$, (b) $s_{ubs}/s_{ut} = 0.6$

the cavity above the advancing spudcan. Results for weightless soil may be applied in practice, by augmenting the bearing resistance by the appropriate surcharge contribution (equation (4)), but only for penetrations down to the maximum cavity depth ($d \leq H$). The depth of back-flow was substantially greater than for a single layer clay, increasing as the strength ratio reduced for a given layer thickness ratio. A convenient expression was proposed to estimate the cavity depth above spudcans penetrating in stiff-over-soft clay.

For $s_{ubs}/s_{ut} \geq 0.6$, failure occurred mainly through a combination of general shear (i.e. classical failure mechanism with surface heave) and partial punching shear modes. In contrast, for $s_{ubs}/s_{ut} < 0.6$, soil displacements were predominantly directed vertically downwards, becoming more confined laterally, and with deeper extent of plastic yielding into the lower layer, as the strength ratio decreased. Severe punch-through was associated with purely vertically downwards soil displacements beneath the spudcan in the upper layer and consequently a soil plug with the shape of an inverted truncated cone eventually forced down into the

underlying soft layer. This failure mode would cause the most catastrophic failure of the supported jack-up unit.

The potential for punch-through was found for all cases where the strength ratio $s_{\text{ubs}}/s_{\text{ut}} \leq 0.6$, with the severity being greater for lower strength ratio. Further, for a critical strength ratio, the degree of bearing pressure reduction with penetration, that is the severity of punch-through, increased with increasing relative thickness of the upper layer (t/D).

Punch-through was still found to occur for higher strength ratios, $s_{\text{ubs}}/s_{\text{ut}} \geq 0.6$, for moderately thick crust layers ($t/D \geq 1$) regardless of normalised shear strength, $s_{\text{ubs}}/\gamma_b D$, where soil back-flow occurred well within the upper layer. In those cases, reduction of bearing resistance occurred very gradually after initiation of soil back-flow, at the point where the spudcan was close to penetrating through the top layer. By contrast, for higher normalised strength of $s_{\text{ubs}}/\gamma_b D \geq 0.36$, severe punch-through was triggered for strength ratios $s_{\text{ubs}}/s_{\text{ut}} < 0.6$, at relatively shallow penetrations well before the layer interface and regardless of the relative crust thickness, with the severity increasing with increasing normalised strength. For these cases, soil back-flow was delayed until the spudcan was within the lower layer.

Positive strength gradients in the underlying layer, expressed in terms of the non-homogeneity ratio kD/s_{ubs} , had the greatest impact on the bearing resistance for thin crusts (low t/D). The potential for punch-through was restricted to $s_{\text{ubs}}/s_{\text{ut}} \leq 0.4$, and even then the severity of punch-through reduced markedly with increasing kD/s_{ubs} . While severe punch-through occurred for $kD/s_{\text{ubs}} = 0$, very mild or even no reduction in bearing resistance was found as kD/s_{ubs} approached unity.

The relative roughness of the spudcan base was found to have little influence on the penetration response, mainly because of the plug of strong clay carried down with the spudcan, which limited the amount of shearing at the spudcan–soil interface.

The design approach recommended in the offshore design guidelines (SNAME, 2002) provided a very poor estimate of the penetration resistance profile, with significant underestimation of bearing capacity and inaccurate identification of the likelihood and severity of punch-through. This underlines an urgent need to reassess the SNAME approach and to develop a more rational calculation method. The results from this study have led to a new mechanism-based design approach (Hossain & Randolph, 2009b).

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NOTATION

A	spudcan plan area at largest section
D	foundation diameter at largest section
d	penetration depth of spudcan base (lowest point of largest section)
H	open cavity depth after spudcan installation (taken as identical to back-flow depth)

k	rate of increase of undrained strength with depth within the lower clay layer
N_{cmax}	maximum bearing capacity factor
q_{net}	net bearing pressure
q_{net0}	net bearing pressure in weightless soil
s_{ub}	undrained shear strength of bottom layer soil
s_{ubs}	undrained shear strength of bottom layer soil at the layer interface
s_{ut}	undrained shear strength of top layer soil
t	thickness of top layer
z	depth below the soil surface
γ_b'	effective unit weight of bottom layer soil
γ_t'	effective unit weight of top layer soil

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