Effect of installation on the bearing capacity of a spudcan under combined loading in soft clay

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Abstract

Previous numerical analyses of the bearing capacity of an embedded footing under combined vertical, horizontal and moment loading have been based on the assumption that the foundation is ‘wished in place’, surrounded by undisturbed soil. Under these conditions the large displacement and remoulding of the soil during the footing installation are not accounted for. This assumption results in an overestimation of the capacity. This paper presents results of the size and shape of the bearing capacity surface of a spudcan footing in soft clay that account for the effects of installation in the modelling. Results for soil sensitivities between one and five and for embedment depths up to three diameters are provided. The findings of this study have practical application in the site-specific assessment of mobile jack-up platforms in soft clay.

1 Introduction

The behaviour of foundations subjected to combined vertical (V), horizontal (H) and moment (M) loading is a fundamental problem faced by geotechnical engineers (see Figure 1 for the sign conventions of positive loads and associated displacements adopted in this paper). Spudcan foundations for offshore mobile jack-up drilling platforms are an example of the type of foundations that are often subjected to combined VHM loading conditions. For site-specific assessments of jack-up rigs, the bearing capacity surface under combined loading is used to evaluate the stability of the spudcan foundations under design storm loading (SNAME 2008; ISO 2012). During installation and preloading, spudcans can penetrate up to three diameters in soft clay (Menzies and Roper 2008) and soil can flow around the advancing spudcan, embedding the footing in the backflow (Hossain et al. 2005; Hossain and Randolph 2009a). The backflow soil, though remoulded during the installation process, is widely perceived to provide additional bearing capacity, especially for moment and

horizontal loading, when compared with a spudcan in stiff clay with no backflow. However, the perceived benefits need to be validated and quantified before they can be used in jack-up site assessment.

Figure 1. A spudcan footing under combined VHM loading and corresponding displacements (LRP: Loading Reference Point)

Finite element studies aimed at developing the *VHM* capacity surfaces for embedded spudcans have been reported by Templeton et al. (2005), Templeton (2009) and Zhang et al. (2011; 2012). Enlarged surfaces based primarily on Templeton's work have also been incorporated in the ISO 19905-1 guidelines for site-specific assessment of jack-up platforms (ISO 2012). However, in the studies mentioned, as in most numerical studies of embedded objects under combined loading (e.g. O'Neill et al. 2003; Elkhatib 2006; Tian et al., 2013 for plate anchors; Gourvenec 2008 for shallow foundations), the spudcan foundations were all assumed 'wished in place' and the bearing capacity was calculated with an undisturbed soil profile. This is not realistic because the footing installation alters the strength profile significantly through both the large displacement of soil and strength softening due to remoulding. This has been demonstrated by, amongst others, Zhou and Randolph (2007; 2009), Hossain and Randolph (2009a; 2009b) and Gan et al. (2012). The resulting effect of the wished in place assumption is that the capacity of the footing is overestimated. This has been confirmed by comparing the results from geotechnical centrifuge experiments with

numerical results for a similar spudcan footing in a normally consolidated (NC) soft clay (Zhang et al. 2013). Therefore, it is the aim of this paper to use a realistic post-installation soil profile in the numerical model to predict the bearing capacity of a spudcan footing accurately.

In this study, the continuous spudcan penetration process was simulated using an axisymmetric large deformation finite element (LDFE) approach with a strain softening soil model. The post-installation soil strength profile was then mapped onto a three-dimensional mesh and the capacity of the spudcan under combined loading was subsequently calculated using displacement-controlled small strain finite element (SSFE) analyses (as only small displacement excursions are required to mobilise the capacity once the spudcan is installed). This numerical strategy allowed the influence of spudcan installation on the bearing capacity of the foundation to be quantified.

The purpose of this study is to complement the centrifuge tests of Zhang et al. (2013) and expand the data base to cover a wider range of soil properties and embedment depths. The bearing capacity surface of a typical spudcan in soft clay for soil sensitivities up to five and embedment depths up to three diameters are reported.

2 Overview of previous analyses that neglected the installation process

Zhang et al. (2011) presented a numerical investigation of the combined bearing capacity of spudcan footings in normally consolidated soft clay using the wished in place method. The bearing capacity of the spudcan was expressed as a surface in the *VHM* load space:

$$f = c_1 \left(\frac{|H|}{h_0 V_{ult}} \right)^{2.5} + c_2 \left(\frac{|M/D|}{m_0 V_{ult}} \right)^{1.5} - \frac{2c_1 c_2 e H M/D}{h_0 m_0 V_{ult}^2} + \left(\frac{V}{V_{ult}} \right)^2 - 1 = 0 \quad (1)$$

where V_{ult} is the ultimate capacity of the spudcan under pure vertical loading, D is the diameter of the spudcan, h_0 and m_0 are size parameters that specify the intercepts of the surface with the H and M/D axis, respectively, as fractions of the pure vertical capacity, and c_1 and c_2 are shape parameters used in the VH and VM planes, respectively. The parameter e describes the eccentricity of the HM cross-sections of the surface. It is related to the vertical load level v ($v = V/V_{\text{ult}}$) by $e = e_1 + e_2 v^2$, where e_1 corresponds to the eccentricity of HM cross-section at $v = 0$; whereas e_2 describes the variation of eccentricity with vertical load level.

The analyses by Zhang et al. (2011) included a parametric study on the influence of embedment ratio w/D (Figure 1). The size of the surface normalised by V_{ult} was found to expand with embedment depth. This is most obvious in the H axis, as h_0 increases considerably with w/D ; whereas m_0 is relatively constant for different depths.

3 Numerical approach

3.1 LDFE simulation of spudcan penetration

The spudcan penetration was simulated using an LDFE approach termed the ‘Remeshing and Interpolation Technique using Small Strain’ (RITSS) method, originally proposed by Hu and Randolph (1998). The entire penetration process is divided into numerous small steps, and the deformed soil geometry is re-meshed at the commencement of each step to avoid mesh distortion. In essence, the RITSS method is a decoupled arbitrary Lagrangian Eulerian approach with an updated Lagrangian calculation in each step and the ‘convection’ of the stress and material properties from the old to the new mesh (Randolph et al. 2008). In this paper the RITSS strategy was coupled with the commercial software Abaqus/Standard (Dassault Systèmes 2011). The main program calling Abaqus to generate the mesh and

implement updated Lagrangian calculation in each step is coded in Fortran. Two Python files are written beforehand to establish the finite element model and to extract data from the result files automatically. Python is the built-in script language of Abaqus (Dassault Systèmes, 2011). A detailed description of the method and its implementation in Abaqus can be found in Wang et al. (2010a; 2010b; 2013). For numerical efficiency the spudcan penetration was simulated as an axisymmetric problem.

To consider the soil disturbance during the installation process and the subsequent impact on the combined bearing capacity of the spudcan, a modified Tresca model that describes the strain softening and strain rate dependent behaviour of clay under undrained conditions (Einav and Randolph 2005) was incorporated into the LDFE analysis. In the model the undrained shear strength of soil is reduced by accumulated plastic shear strain and enhanced if the shear strain rate is higher than a reference strain rate. Though simple, the model has proven useful for solving practical problems and has been used in an increasing number of applications, such as spudcan penetration (Hossain and Randolph 2009b; Tho et al. 2012), penetrometer tests (Zhou and Randolph 2009), pipeline vertical embedment and lateral buckling (Hu et al. 2010; Wang et al. 2010b; Chatterjee et al. 2012) and submarine landslides (Wang et al. 2013b). In the current work, the rate effect was not taken into account as it has been argued that the strain rate experienced by the soil in the field during spudcan penetration is similar to the typical laboratory shearing rates (Hossain and Randolph 2009b). The focus therefore was placed on capturing the soil remoulding (strain softening) caused by spudcan installation.

The mathematical form of the soil model adopted is as follows (Einav and Randolph, 2005):

$$s_u = s_{u_i} \left[\delta_{\text{rem}} + (1 - \delta_{\text{rem}}) e^{-3\xi/\xi_{95}} \right] \quad (2)$$

where s_{ui} is the intact shear strength prior to any softening, δ_{rem} is the inverse of sensitivity S_t , ξ is the accumulated absolute plastic shear strain, and ξ_{95} is the value of ξ for the soil to undergo 95% of the full level of remoulding, thus reflecting the relative ductility of the soil. The value of ξ_{95} cannot be measured directly in soil laboratory tests, instead, it is usually determined by fitting the resistance profile of penetrometer tests. Typical values of ξ_{95} have been estimated in the range of 10 to 50 (i.e., 1000% to 5000% shear strain) by comparison with cyclic penetration and extraction data (Randolph 2004; Einav and Randolph 2005). Eq. 2 suggests that the soil strength is diminished according to an exponential function as the soil state changes from intact to fully remoulded.

When the strain-softening model was incorporated into the Abaqus-based RITSS method, both the accumulated absolute plastic shear strain and stresses at old integration points were extrapolated to old element nodes through the popular Superconvergent Patch Recovery (SPR; Zienkiewicz and Zhu, 1993) technique, followed by interpolation from the old element nodes to new integration points. The accuracy of SPR was verified thoroughly in previous studies (e.g. Wang et al. 2010b; Wang et al. 2013). The strain localization may become severe with increasing soil sensitivity, which leads to numerical instability. However, given the soil sensitivity is not too large, for instance $S_t < 5$, the numerical stability can be ensured as shown later.

In the LDFE analyses, the intact soil strength (s_{ui}) was assumed to increase with depth at a strength gradient k of 1.2 kPa/m (Figure 2). The mudline strength was idealised to be zero, though a small value of 0.1 kPa was used for numerical reason. This represents a typical strength profile for offshore normally consolidated soft clay. ξ_{95} was assigned an intermediate

value of 30. Young's modulus was taken as $E = 500s_u$, and Poisson's ratio was assumed to be 0.49 to approximate undrained conditions. The submerged unit weight was 6.5 kN/m^3 .

A frictional spudcan–soil interface was considered in the penetration analyses. It was modelled as hard contact following Coulomb law with a friction coefficient of 0.3. By considering arbitrary relative separation and sliding between master surface (spudcan) and slave surface (soil), the finite-sliding tracing approach, rather than the conventional small-sliding one, was adopted. A shear stress limit of $0.3s_{u0}$ was prescribed at the interface, where s_{u0} is the intact shear strength at the current penetration depth, measured from the original soil surface to the load reference point (*LRP* as shown in Figure 2) at the commencement of each small step. The adopted shear stress limit is close to that measured between the caisson wall and soil in the centrifuge by Chen and Randolph (2007). It is worth noting that the spudcan roughness has limited influence on the penetration resistance (Hossain and Randolph 2009a). A smooth cylindrical shaft of $1/3$ of the spudcan diameter was included in the LDFE model. This represented the jack-up leg and was the same dimension as the physical scaled model used in the centrifuge tests of Zhang et al. (2013). In the LDFE penetration analysis this smooth shaft was implemented by a roller boundary condition that prohibits the lateral movement of the soil.

In the LDFE analysis, the vertical and horizontal limits of the soil domain were taken to be $6D$ and $8D$, respectively. The soil was meshed with quadratic triangular elements (CAX6 in Abaqus), and the spudcan was idealised as rigid. The soil around the spudcan was discretised with a finer mesh, with a typical element size of $0.02D$. The incremental penetration of the spudcan in each step was selected as $0.005D$, which was sufficiently small to avoid penetration-induced excessive distortion of soil elements.

Figure 2. Spudcan and intact soil strength profile

Figure 3 shows a comparison of the LDFE simulation with the centrifuge data reported in Zhang et al. (2013) (for $S_t = 2.2$ as derived from cyclic T-bar tests, a similar NC soil profile and an identical spudcan shape). Evidently, the LDFE successfully captures the vertical penetration resistance observed in the centrifuge. At shallow depths, the high bearing capacity factor is due to the high local heterogeneity of the soil (kD/s_{u0} , where s_{u0} is the intact shear strength at the current depth of penetration, Figure 2). The bearing capacity factor decreases rapidly with depth and becomes almost constant at a value of approximately 10 at depths greater than $0.5D$. This represents the formation of a localised flow-around failure mechanism. The results from the wished in place SSFE analyses of Zhang et al. (2011) clearly overestimate the bearing capacity at depth.

Figure 3. Comparison of LDFE results against centrifuge tests and SSFE results

Figure 4 shows the undrained shear strength profile after the spudcan has penetrated to a depth of $1.5D$. The right half of the figure shows the profile for the soil with $S_t = 2.2$, while the left half of the figure shows the profile for an ideal condition with $S_t = 1$ (i.e. no softening induced in the soil). Beyond the influence zone of the spudcan penetration, the strength contour represents a normally consolidated soil profile, where s_u increases linearly with depth. Clearly, the spudcan penetration disturbs the soil strength distribution around the spudcan considerably. For $S_t = 1$, the soil strength around the footing changes from the intact condition due to the penetration-induced soil displacement. Softer material is dragged down from the top region surrounding the spudcan. If strain softening is further considered, the soil strength around the spudcan becomes even weaker, as shown for $S_t = 2.2$. For both cases

shown in Figure 4, the influence zone of the spudcan penetration is contained horizontally within $1D$ from the spudcan centre. This is consistent with the findings from large deformation simulations using the coupled Eulerian-Lagrangian method (Hu et al. 2012; Tho et al. 2012).

Figure 4. Soil strength profile after spudcan penetration

3.2 SSFE analysis of spudcan combined bearing capacity

After the spudcan was penetrated to the target embedment depth, the remoulded soil strength profile of the LDFE mesh was mapped to a three-dimensional mesh for SSFE analyses that investigate the combined bearing capacity surface. Because the post installation shear strength profile is axisymmetric, a point in the three dimensional SSFE mesh has equal shear strength to a point in the LDFE mesh that has the same vertical depth and radial distance from the centre line of symmetry. During mapping, the corresponding position in the two-dimensional mesh for each integration point of the three-dimensional mesh was searched and the current soil strength at three-dimensional integration point was interpolated in the quadratic triangular element that contained the point. The subsequent SSFE analyses were performed using Abaqus/Standard.

Similar to Zhang et al. (2011), constant displacement ratio probes were performed to establish the *VHM* bearing capacity surface. In a probe test, a fraction of the ultimate vertical load is applied to the footing as a direct force, while horizontal and rotational displacements of a fixed ratio ($u/D\theta$ is constant during a test) are applied to the spudcan until the load state does not change with any further displacement. A probe test determines a single point on the *VHM* capacity surface. For the results in this paper, each *HM* cross-section of the *VHM* surface was determined by nine probe tests and cross-sections at five different vertical load levels (0, 25%, 50%, 75% and 90% of ultimate vertical load) were calculated. Together, these establish the complete *VHM* surface.

The soil in the SSFE simulations was also considered to be a Tresca material, but because of the small movements of the spudcan, strain softening was not considered. In all SSFE analyses, a constant E/s_u ratio of 10,000 and Poisson's ratio of 0.49 were assumed. The artificially high E/s_u ratio limited mesh distortion and reduced computation time without affecting the calculated ultimate capacity (Zhang et al. 2011; Zhang et al. 2012b).

The SSFE analyses were performed in three-dimensional space due to the combined *VHM* loading conditions. The element type (C3D8 in Abaqus) and the meshing principles are all similar to Zhang et al. (2011), except that the element size immediately surrounding the spudcan was limited to $0.02D$, compared to $0.005D$ in Zhang et al. (2011). This matched the element size used in the LDFE analyses to ensure accurate mapping of the soil strength (noting that the element size used in the LDFE analyses cannot be smaller because of numerical instability). The slightly coarser mesh results in an approximately 3% over-estimation in all of the vertical, horizontal and moment capacities (compared to the finer mesh used in the wished in place analyses of Zhang et al. (2011)).

Similar to the LDFE analyses, the shaft of the jack-up leg was represented by a smooth boundary condition that did not permit lateral movement. However, during the probe tests, the shaft was assumed to be stationary while the spudcan was subjected to displacement excursions. Therefore the calculated V , H , M reactions are related to the soil resistance mobilised only by the spudcan and they were measured at the *LRP*.

4 Results

A parametric study of the combined *VHM* bearing capacity surface of a typical spudcan in a normally consolidated clayey soil was performed for the following conditions.

- Soil sensitivity $S_t = 1, 2.2, 3, 4$ and 5 . The value of 2.2 was chosen to match the centrifuge tests of Zhang et al. (2013). The range of S_t was chosen according to Kvalstad et al. (2001), Andersen and Jostad (2004) and Randolph (2004) who claim the sensitivity of typical marine clay to be 2 to 5 .
- Embedment ratio $w/D = 0.5, 1, 1.5, 2$ and 3 .

4.1 Vertical penetration resistance

Figure 5 presents the relationship between the vertical bearing capacity factor ($N_c = V/A_{s_{u0}}$) and embedment depth for the spudcan in soils with different sensitivities (note for $S_t = 5$, only the results for depths up to $1.5D$ are shown as the computation became increasingly unstable at larger embedments). The results from the wished in place SSFE analyses (Zhang et al. 2011) and retrospective predictions using the method of Hossain and Randolph (2009a; 2009b) are shown for comparison. As expected, the N_c factors predicted by the LDFE analyses reduces with increasing soil sensitivity. Although for the ideal soil with $S_t = 1$, i.e. without strength degradation, the calculated N_c factor by LDFE is considerably smaller than

the wished in place results because of softer material being dragged down. This alters the soil strength around the footing, as illustrated in Figure 4. For soil with S_t greater than 1, strain softening further reduces the bearing capacity factor. However, the magnitude of reduction decreases with increasing sensitivity, and no significant difference is observed between $S_t = 4$ and 5.

Figure 5. Normalised vertical penetration resistance

After the initial reduction at shallow depth, the wished in place analyses suggest that the N_c factor will increase considerably with depth until reaching the plateau from $w/D = 1.5$ due to the formation of a localised flow-around mechanism. Although the LDFE results for $S_t = 1$ show a similar trend, the steady state N_c is lower and the increase before $w/D = 1.5$ is comparatively insignificant. For soil with $S_t = 2.2, 3, 4$ and 5, LDFE analyses indicate

1 negligible change of the N_c factor from an embedment of $0.5D$. The different patterns of
2 response between the wished in place and LDFE results are due to soil displacement and
3 strain softening, which reduces the soil strength in the LDFE simulation. The failure
4 mechanism tends to be confined in the disturbed soil around the footing rather than extending
5 to the undisturbed stronger soil. Therefore, the localised flow-around mechanism forms at a
6 much shallower depth. Figure 6 shows a comparison of the soil flow mechanisms at $w/D =$
7 0.5. For the wished in place case, a shallow mechanism extending to the soil surface is found,
8 whereas, in the LDFE simulation, a localised flow-around mechanism is already formed.
9 (Note the spudcan geometry was the same in both analyses, though it appears shorter in the
10 right hand side plot due to the discretisation of the shaft, as shown in Figure 2.)

11 SSFE, wished-in-place LDFE, $S_t = 4$
12 **Figure 6. Comparison of soil flow mechanism at $w/D = 0.5$**

13
14 Hossain and Randolph (2009a and b) proposed the following equations to calculate the
15 bearing capacity factor in clay based on LDFE analyses:

$$16 \quad N_c = \left(\delta_{\text{rem}} + (1 - \delta_{\text{rem}}) e^{-3\zeta_b/\zeta_{95}} \right) N_{c_ideal} \quad (3)$$

17 where

18 δ_{rem} and ζ_{95} are as defined earlier;

ξ_b stands for the average shear strain as the soil transverses the flow around mechanism, and is recommended to be 2.4; and

N_{c_ideal} is the spudcan bearing capacity factor in ideally non-strain softening soil ($S_t = 1$) and is calculated by

$$N_{c_ideal} = 10(1 + 0.065 \frac{w}{D}) \leq 11.3 \quad (4)$$

Figure 5 shows that the Hossain and Randolph method predicts the response well for depths greater than $2D$. However, the method underestimates the capacity for shallower depths. To consider the effect of soil sensitivity on vertical capacity, the following reduction factor is defined:

$$R_v = \frac{N_c}{N_{c_ideal}} \quad (5)$$

where N_c is the steady state bearing capacity factor mobilised at depth greater than $0.5D$ for soils with $S_t > 1$. N_{c_ideal} is the derived bearing capacity factor for $S_t = 1$ and for simplicity is taken as 11. Figure 7 presents the variation in the reduction factor with soil sensitivity. A simple expression of S_t was found to fit the data well:

$$R_v = \begin{cases} S_t^{(-0.1)}, & S_t \leq 4 \\ 0.87, & 4 < S_t \leq 5 \end{cases} \quad (6)$$

Figure 7. Reduction factor for vertical bearing capacity

4.2 Failure envelopes in the VH ($M = 0$) and VM ($H = 0$) planes

This section and the section below present the results of combined bearing capacity calculated from the SSFE analyses after taking account of the spudcan installation effects.

Figure 8 presents failure envelopes of the spudcan in the VH ($M = 0$) and VM ($H = 0$) planes at an embedment ratio (w/D) of 1.5 and for different soil sensitivities. The failure envelopes from wished in place analyses of Zhang et al. (2011) are included for comparison. The figure demonstrates that the capacity of the footing was considerably over-estimated by the wished in place analyses. By merely including the soil movement during spudcan installation, which is the $S_t = 1$ case, the size of the failure envelopes is significantly reduced. When strain softening of the soil is considered (cases with $S_t > 1$), the size of the failure envelopes reduces further. As expected, the size of the failure envelope decreases with increasing soil sensitivity. However, the influence of sensitivity reduces with increasing S_t . The failure envelopes for $S_t = 5$ and $S_t = 4$ are very similar. For this reason, no further numerical calculations were performed for $S_t = 5$.

(a)

(b)

Figure 8. Failure envelopes normalized by soil strength for different S_t ($w/D = 1.5$): (a) VH ($M = 0$) plane; (b) VM ($H = 0$) plane

To illustrate the effect of embedment, the failure envelopes in the VH ($M = 0$) and VM ($H = 0$) planes at different embedment depths are presented in Figure 9 for the example of $S_t = 3$. With increasing embedment depth, the vertical bearing capacity factor remains approximately constant. This is consistent with the LDFE results presented in Figure 5 (though there are small numerical differences (of the order of 2-3%) on the exact value between SSFE and

LDFE analyses due to different element families). However, the horizontal and moment bearing capacity factors (H_{ult}/As_{u0} and M_{ult}/As_{u0} , represented by the intercepts of the envelopes with the H axis and M axis respectively) increase with embedment. This results in the expansion of the failure envelope in the H and M dimensions with depth. The results for other sensitivities follow a similar trend. In Figure 9, the results from wished in place analyses (Zhang et al. 2011) are also included for comparison. For the same embedment depth, the envelope obtained from wished in place analyses is considerably larger, indicating over-estimation of the bearing capacity as installation effects are neglected. Besides that, in the wished in place analyses all of the vertical, horizontal and moment bearing capacity factors increase with depth. This is different for the current results accounting for installation effects, which shows the vertical capacity factor remaining almost constant with depth while horizontal and moment bearing capacity factors increase.

(a)

(b)

Figure 9. Failure envelopes at different embedment depths ($S_t = 3$): (a) VH ($M = 0$) plane; (b) VM ($H = 0$) plane

Figure 10 shows the ratios of H_{ult}/V_{ult} and M_{ult}/DV_{ult} for all the combinations of embedment depths and soil sensitivities. Again the results from wished in place analyses are provided for comparison. The pattern of change with embedment depth of the ratio of H_{ult}/V_{ult} , is similar to the wished in place results. From $w/D = 0.5$ to 1.5, it increases in a linearly fashion with depth. However, from $w/D = 1.5$, the rate of increase reduces gradually. In contrast to a constant value suggested by the wished in place results, the current analyses indicate that M_{ult}/DV_{ult} increases with embedment depth. The patterns among different sensitivities are broadly similar, but differ in depth between $w/D = 0.5$ to 1, with larger increases for less sensitive soil. As illustrated, the ratios of H_{ult}/V_{ult} for all combinations of w/D and S_t , after considering the installation effects, are smaller than the corresponding values calculated with the wished in place assumption. This indicates that the installation has a greater impact on the horizontal capacity than the vertical capacity. However, the same does not hold for the moment capacity.

(a)

(b)

Figure 10. H_{ult}/V_{ult} and M_{ult}/DV_{ult} ratios for all w/D and S_t combinations

4.3 Bearing capacity surface in the combined VHM space

The above section has reported envelopes of the spudcan under the combined VH and combined VM loading. These are the cross-sections of the VHM failure surface on $M = 0$ and $H = 0$ planes respectively. This section details the complete failure surface under combined VHM loading in the form of HM cross-sections in the planes of $V/V_{ult} = 0, 0.25, 0.5, 0.75$ and 0.9 .

Figure 11 shows an example of HM cross-sections at different vertical load levels calculated for $S_t = 2.2$ at embedment ratios of $0.5, 1, 1.5, 2$ and 3 . The results are normalised by V_{ult} at the respective embedment depth. Each circular dot represents a data point calculated from a probe test. The dashed curves are cross-sections of a fitted surface which will be explained later. Note that only the data points in the $(+H, +M)$ and $(-H, +M)$ quadrants were calculated. However, as a load combination (V, H, M) has the same effect as a combination of $(V, -H, -M)$, those calculated data points are also plotted in the diagonal opposite quadrants for presentation.

The size of the cross-sections decreases with increasing vertical load level for each suite of HM cross-sections at a specified depth. The eccentricity also reduces with increasing vertical load. This is most obvious at depth of $0.5D$. Comparison of the results at the different depths suggests that the overall eccentricity of the HM cross-section reduces with increasing embedment depth. However, the normalised size of the HM cross-sections at the same V/V_{ult} ratio increases for the same change of embedment depth. These observations are very similar to those identified from the wished in place analyses (Zhang et al. 2011).

(a) $w/D = 0.5$

(b) $w/D = 1$

(c) $w/D = 1.5$

(d) $w/D = 2.0$

1

(e) $w/D = 3$

Figure 11. VHM failure surface at different embedment depths ($S_t = 2.2$)

4.4 Analytical expression of the failure surface

The simplified Eq. 1 with the shape parameters c_1 and c_2 both assigned a value of unity, as shown in Eq. 7, was used to fit the current numerical results.

$$f = \left(\frac{|H|}{h_0 V_{ult}} \right)^{2.5} + \left(\frac{|M/D|}{m_0 V_{ult}} \right)^{1.5} - \frac{2eHM/D}{h_0 m_0 V_{ult}^2} + \left(\frac{V}{V_{ult}} \right)^2 - 1 = 0. \quad (7)$$

While m_0 was conveniently determined as M_{ult}/DV_{ult} , the parameters h_0 , e_1 and e_2 were determined by least squares regression. The optimized h_0 values are slightly larger than H_{ult}/V_{ult} ratios, which improves the fit at relatively shallow depths where the eccentricity of the surface on HM plane is large. Table 1 summarises the best-fit parameters. The quality of approximation is shown in Figure 10 for the example $S_t = 2.2$.

Figure 12 illustrates the variation of the eccentricity parameters with S_t and w/D (changes in h_0 and m_0 follow a similar pattern to Figure 10). It can be seen that the parameter e_1 , which represents the eccentricity of the *HM* cross-section for $V = 0$, decreases with embedment depth, but increases with soil sensitivity. In absolute values, the parameter e_2 , which is used to describe the change of eccentricity of the *HM* cross-section with vertical load level, shows a similar trend to e_1 .

Table 1. Summary of curve fitting parameters

S_t	Depth (w/D)	h_0	m_0	e_1	e_2
1	0.5	0.209	0.120	0.534	-0.444
	1	0.256	0.126	0.293	-0.205
	1.5	0.311	0.136	0.090	-0.066
	2	0.341	0.141	0.012	-0.030
	3	0.365	0.143	-0.040	0.021
2.2	0.5	0.191	0.118	0.586	-0.463
	1	0.231	0.122	0.400	-0.262
	1.5	0.278	0.131	0.246	-0.122
	2	0.300	0.135	0.138	-0.091
	3	0.321	0.138	0.072	-0.047
3	0.5	0.188	0.117	0.593	-0.459
	1	0.224	0.120	0.443	-0.276
	1.5	0.261	0.128	0.275	-0.148
	2	0.286	0.132	0.196	-0.102
	3	0.312	0.136	0.113	-0.051
4	0.5	0.185	0.117	0.607	-0.449
	1	0.217	0.117	0.474	-0.274
	1.5	0.243	0.124	0.339	-0.180
	2	0.265	0.128	0.260	-0.135
	3	0.289	0.132	0.169	-0.080

Figure 12. Variation of shape parameter e_1 and e_2 with S_t and w/D

5 Comparison with centrifuge data

Zhang et al. (2013) reported results of combined loading tests in normally consolidated kaolin clay in a geotechnical drum centrifuge. The experiments were performed with the same spudcan shape, a similar soil strength profile and a soil sensitivity of 2.2. The *VHM* capacity surface was established in the experiments by first penetrating the model spudcan to the target embedment depth before applying an excursion of horizontal or/and rotational displacement(s) whilst holding the vertical displacement constant. These tests are colloquially known as ‘swipe’ tests (other examples include Tan 1990; Gottardi et al. 1999; Martin and Houlsby 2000; Cassidy et al. 2004; Bienen et al. 2006; Cassidy 2007; Hodder and Cassidy, 2010) and it is argued that they track a path on the *VHM* surface corresponding to the current embedment depth.

Using the experimental results, Zhang et al. (2013) presented a *VHM* bearing capacity surface for embedment ratios of $0.7D$ and $1.45D$ of

$$f = \left(\frac{H}{h_0 V_{\text{ult}}} \right)^2 + \left(\frac{M/D}{m_0 V_{\text{ult}}} \right)^2 - \frac{2eHM/D}{h_0 m_0 V_{\text{ult}}^2} - \left(\frac{4}{(1+\chi)^2} \right)^2 \left(\frac{V}{V_{\text{ult}}} + \chi \right)^2 \left(1 - \frac{V}{V_{\text{ult}}} \right)^2 = 0 \quad (8)$$

where χ represents the ultimate tensile capacity as a proportion of the vertical compressive capacity (with 0.6 fitting the experimental results in kaolin clay). The remaining parameters have the same meanings as defined for Eq. 7. The eccentricity parameter e was assumed to be constant for a specific embedment ratio.

The major difference between the numerically derived surface of Eq. 7 and the experimentally derived surface of Eq. 8 is the assumption of tensile capacity, with the numerical assumption of a fully bonded soil-foundation interface and the Tresca failure criterion creating the same tensile and compressive capacities. To fit the respective results the numerical surface uses an ellipse in *VH* and *VM* planes whereas the experiments were best fit with a parabola. Whilst the form of equation slightly modifies the shape of the envelope, the values of h_0 and m_0 determine the size of the envelope and these are compared here. Figure 13 shows the numerical and experimental h_0 and m_0 values for the soil condition of $S_t = 2.2$. It is demonstrated that the trends of the numerical and experimental results compare reasonably well, though the numerical surface sizes are slightly larger.

Figure 13. Comparison of h_0 and m_0 values between numerical and experimental results

Figure 14 compares the eccentricity of the HM cross-sections between the numerical and experimental results. Note that the parameter e_1 (Eq. 7) represents the eccentricity of the HM cross-section at $V/V_{ult} = 0$. The numerical results indicate the eccentricity will reduce with increasing vertical load, as indicated by negative e_2 values (listed in Table 1). However, the experimental formulation uses a constant eccentricity (e) at all vertical load levels; therefore e represents an average value for different vertical load levels. Figure 14 clearly shows a similar trend between the numerical and experimental results (though again with the experimental results being lower).

Figure 14. Comparison of experimental and numerical eccentricity

6 Conclusions

Results on the size and shape of the combined bearing capacity surface of a typical spudcan in normally consolidated soft clay are presented. These were obtained from numerical analyses that explicitly consider the effects of spudcan installation. This has not been done before. The results cover a range of soil sensitivities up to five and embedment depth up to three footing diameters. Based on the numerical results, it is observed that:

- i) Penetration of a spudcan in normally consolidated clay will be governed by a localised flow around mechanism from a depth of approximately $0.5D$, beyond which the bearing capacity factor N_c remains approximately constant. A simple expression is proposed to describe the influence of soil sensitivity on the vertical capacity.

- ii) The large displacement and strength softening due to the installation of the footing significantly reduces its combined bearing capacity compared to that of a wished in place foundation. As soil sensitivity increases, the capacity reduces, but at a decreasing rate. The footing has approximately the same capacity in soil with $S_t = 5$ as $S_t = 4$.
- iii) The normalised size of the combined bearing capacity surface increases with embedment while the eccentricity of the surface reduces. An analytical expression is proposed that fits the bearing capacity surface at different depths well.
- iv) The numerical results compare favourably with the centrifuge experimental results, with all the main features of the centrifuge results successfully captured.

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