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Abstract

A macro-element model for predicting the load-displacement behaviour of a spudcan foundation in clay overlying sand when subjected to combined vertical, horizontal, and moment loading is introduced. Observations from detailed drum centrifuge tests that measured the effect of the underlying sand layer on the foundation behaviour are combined with finite-element results and theoretical developments to derive the components of the model. The yield surface defined by the centrifuge test results suggests that as the spudcan nears the underlying sand layer, the absolute horizontal capacity remains relatively constant, while the vertical and moment capacities increase at approximately the same normalised rate. The model is demonstrated to accurately predict foundation behaviour by retrospectively simulating the experimental results. This macro-element model has the advantage that it can be integrated into the structural analyses of jack-up platforms required for site-specific assessments.

Keywords: bearing capacity; centrifuge modelling; footings/foundations; offshore engineering; soil/structure interaction
INTRODUCTION

Mobile jack-up platforms are used offshore for drilling and for the installation and maintenance of wind turbines (Zaaier and Henderson, 2004; Ahn et al., 2017) and hence are frequently relocated. Jack-ups are self-elevating platforms, consisting of a buoyant hull and retractable legs, each attached to a footing called a ‘spudcan’. Spudcans are generally inverted conical in shape with a central spigot and are typically 10-20 m in diameter (Menzies and Roper, 2008).

Environmental loading from wind, waves, and current on the structure apply considerable horizontal ($H$) loads, overturning moments ($M$) and changes in vertical ($V$) loads on the spudcans. The use of plasticity-based macro-element models (Noble Denton & Associates, 1987; Schotman, 1989; Dean, 2010; Randolph and Gourvenec, 2011; Houlsby, 2016) have been demonstrated to be effective for modelling the spudcan-soil interactions required in a site-specific assessment (ISO, 2016) and these models are gradually being implemented in industry design guidelines (ISO, 2016). An advantage is that macro-element models are expressed in the terminology of force components and their work-conjugate displacement components, which is consistent with structural mechanics and can be readily incorporated into numerical modelling programs.

A macro-element model described herein is based on work-hardening plasticity theory and contains four components: (i) a yield surface in combined $VHM$ load space, (ii) a hardening law describing the variation in the size of the yield surface with the change in the vertical plastic penetration, (iii) a description of the elastic behaviour for any load increments within the yield surface, and (iv) a flow rule determining the plastic displacements during an elasto-plastic loading step.

A number of studies have established macro-element models for circular footings or spudcans on clay (e.g., Schotman, 1989; Martin and Houlsby, 2000; Crémer et al., 2001; Martin and Houlsby, 2001; Crémer et al., 2002; Cassidy et al., 2004a; Zhang et al., 2014a), silica sand (e.g., Nova and Montrasio, 1991; Gottardi et al., 1999; Houlsby and Cassidy, 2002; Bienen et al., 2006; Cassidy, 2007; Cheng and Cassidy, 2016), and carbonate sand (e.g., Byrne and Houlsby, 2001; Cassidy et al., 2002). However, these developments are restricted to homogeneous clay or sand soils and do not account for the stratified seabeds often encountered offshore (Baglioni et al., 1982; Menzies and Lopez, 2011). For a soft clay deposit overlying a stiffer sand layer, soil failure involving lateral squeezing of the upper clay
occurs during spudcan installation. Although previous research has reported a substantial increase in the vertical bearing capacity prior to penetration into the sand layer (Dean, 2008; Hossain, 2014; Ullah et al., 2017; Wang et al., 2021), an understanding of behaviour under combined VHM loading is limited. Recently, Wang et al. (2020b) used the finite element method to assess the failure envelope of spudcans in such soil stratigraphy. While additional combined loading capacity was demonstrated and an analytical failure envelope formulated, it has not been verified against experimental data. In this respect, the numerical study may overestimate the ultimate foundation capacity due to the “wished-in-place” assumption that neglects the installation process. Nevertheless, the failure envelope formulation in Wang et al. (2020b) is a convenient basis for developing a macro-element model in this paper, representing the yield surface in the model.

This paper describes geotechnical centrifuge tests designed to develop a macro-element model of the combined load-displacement behaviour for a spudcan in clay overlying sand. These experimental results are combined with results of numerical finite element analyses (Wang et al. 2020b), which may overestimate the ultimate foundation capacity due to the “wished-in-place” assumption that neglects the installation process, and theoretical developments to derive the macro-element model components. The experiments explore the effect of the underlying stronger sand layer on the foundation behaviour, with attention paid to investigating the combined load yield surface. Calibration of the hardening law and flow rule is also carried out, and the predictive capabilities of the model are verified by retrospective simulations of the experimental results.

EXPERIMENTAL PROGRAMME

The tests were conducted in a 1.2 m diameter drum centrifuge (Stewart et al., 1998) at an acceleration of 200 g. The soil sample was contained in a channel with a height of 300 mm and a radial depth of 200 mm, as shown schematically in Fig. 1.

Loading apparatus and spudcan model

Planar loads consisting of VHM loads were applied to the spudcan at a reference point (RP) taken as the midpoint at the base of the widest part of the spudcan. The sign convention for the loads (V, H, M/D) and their work-conjugate displacements (w, u, Dθ) is shown in Fig. 2 (after Butterfield et al., 1997), where D is the spudcan diameter. The thickness of the clay layer between the RP and the sand surface is denoted as T.
Loads were applied using a specially designed loading apparatus (Zhang et al., 2013), which was attached to the actuators on the tool table (Fig. 1). The radial and vertical motions of the actuators applied the vertical and horizontal movements to the footing, respectively. The rotation of the footing was achieved by the differential displacements of two parallel actuators, which were linked by a pivot arm and hinges. This allowed the three degrees of freedom to be controlled independently at the RP and logged by the control software. Fig. 3 shows the assembled loading apparatus.

A model spudcan of $D = 60$ mm was used in this study (12 m at the prototype scale when tested at 200g). The footing diameter was kept the same as that of Zhang et al. (2014a) for comparison with single-layer clay. A cylindrical aluminium shield was used to protect the strain gauges and isolate the soil pressure acting on the loading arm. The gap between the shield and the spudcan was sealed by soft silicone, which subsequently minimises the load transfer between the two. The vertical load was determined directly from the axial strain gauge, while the horizontal load and moment at the RP were transformed from the bending moments measured by the two sets of bending gauges (Fig. 4) via linear bending theory.

**Soil sample and site characterization**

The underlying superfine silica sand (Table 1, after Chow et al., 2019) layer was created by raining sand through water in the channel rotating at an acceleration of 20g (as described in Gaudin et al., 2005). The sand surface was then carefully levelled at 1g with a scraping plate, achieving a thickness of 69 mm. Next, an overlying kaolin clay (Table 2, after Stewart, 1992; Acosta-Martinez and Gourvenec, 2006) layer was prepared by pouring clay slurry with water content twice the liquid limit (120%) through a hose whilst the centrifuge spun at 20g. The nozzle of the hose was submerged in the water but placed high enough above the sand to avoid scouring. The clay was consolidated at 200g with intermittent reductions to 20g to add additional slurry, achieving a normally consolidated clay sample with a depth of 85 mm ($H/D = 1.42$, see notation in Fig. 2). Throughout the tests, the water table was kept at 46 mm above the clay surface.

The relative density $I_D$ of the underlying sand layer was determined from three 60-mm-diameter tube samples taken at equidistant radial locations in the channel after all the tests. An average relative density of 45% was measured, indicating a medium dense sample. The submerged unit weight of the sand $\gamma_s$ was found to be 10.02 kN/m$^3$.
The undrained shear strength profile of the overlying clay layer was determined using a T-bar penetrometer with a projected penetrating area of 100 mm\(^2\) (Stewart and Randolph, 1991; Stewart and Randolph, 1994). This was preferred over a cone penetrometer due to its higher resolution in soft clay (Randolph et al., 1998; Low, 2009), leading to a more accurate interpretation of the bearing capacity factors in the upper clay layer. The tests were conducted at a penetration rate of 1.52 mm/s to ensure undrained conditions (Randolph and Hope, 2004). Fig. 5 shows the undrained shear strength profile interpreted from the T-bar resistance, \(q_{T}\text{-bar}\). A constant T-bar factor \(N_{T}\text{-bar}\) of 10.5 (Stewart and Randolph, 1994) was assumed to derive the undrained shear strength, \(s_u\), based on the relationship \(s_u = q_{T}\text{-bar}/N_{T}\text{-bar}\). The intact \(s_u\) inferred from the first penetration resistance was approximated to vary linearly with depth as

\[
s_u = s_{um} + kz
\]  

(1)

where \(s_{um} = 2.69\) kPa is the shear strength at the mudline and \(k = 1.93\) kPa/m is the shear strength gradient with depth \(z\). However, a sharp increase in the T-bar resistance occurred when the T-bar penetrated within \(~1.2D_{T}\text{-bar}\) (where \(D_{T}\text{-bar}\) is the T-bar diameter) from the sand surface. This is because the soft clay beneath the T-bar was squeezed horizontally as the vertical load began to be borne by the strong underlying sand layer. The average submerged unit weight of the clay \(\gamma'_c\) was 7.32 kN/m\(^3\), obtained from 20-mm-diameter soil samples extracted at different locations in the soil channel.

**Experimental strategy**

The experimental programme consisted of vertical loading tests and swipe tests. Two vertical loading tests involving unloading-reloading loops were conducted to investigate the hardening behaviour and provide information about the vertical elastic stiffness. Displacement-controlled vertical penetration and extraction of the spudcan footing were performed at penetration rates \(\dot{w}\) of 0.127 mm/s corresponding to a non-dimensional velocity \(V = \dot{w}D/c_v\) (where \(c_v\) is the coefficient of consolidation) of 120, ensuring undrained behaviour in the clay (Finnie and Randolph, 1994; Cassidy, 2002).

Swipe tests were used to examine the shape of the yield surface. During the swipe test, the footing was first penetrated to a prescribed vertical displacement, which corresponded to a vertical load of \(V_0\) (defining the maximum vertical load experienced) (amongst many others, Tan, 1990; Gottardi et al., 1999; Martin and Houlsby, 2000; Cassidy, 2007; Govoni et al., 2011). This was followed by a combination of horizontal displacement and rotation while the vertical displacement was kept constant. During the second loading phase, the load path can
be assumed to track along the yield surface, as long as the ratio of the vertical elastic stiffness to the vertical plastic stiffness is large (see, for instance, Martin and Houlsby, 2000) and a correction for the rig flexibility and the soil elasticity is applied (Gottardi et al., 1999).

Twelve swipe tests commencing from two embedment depths (and corresponding $V_0$), with different ratios of horizontal displacement to rotation, were carried out in the one centrifuge sample (Fig. 6, Table 3). The displacement rates were demonstrated to be sufficiently high to allow the modelling of undrained conditions. As the focus of this research is the effects of the underlying sand layer on the yield surface of spudcan foundations in clay, swipe tests were performed at those embedment depths within the zone where the vertical load increased substantially.

**MACRO-ELEMENT MODEL**

**Hardening law**

Fig. 7 shows the vertical load-displacement responses measured during the vertical loading tests and the initial penetration phase of all swipe tests. The consistency between the curves indicates the uniformity of the soil sample. The vertical load $V$ is the bearing capacity of the soil underneath the spudcan, which is given by the vertical force registered by the axial strain gauge, less the soil buoyancy. Initially, the vertical bearing capacity increases linearly with penetration because of the increase in the strength with depth of clay. However, the penetration curve starts to deviate at a depth of $0.4D$ above the sand surface. The bearing capacity increases sharply as the spudcan penetrates towards the sand layer. This is because the softer clay trapped beneath the spudcan base is squeezed horizontally as an increasing portion of the resistance is produced by the stronger underlying sand layer (Wang et al., 2021).

In Fig. 7, the unloading-reloading loop shows that the ratio of the elastic stiffness to the local plastic stiffness is of three orders of magnitude. This is favourable for investigating the yield surface using swipe tests (Martin and Houlsby, 2000) and indeed this ratio is even higher with an increasing elastic stiffness as the spudcan approaches the stronger sand layer (as demonstrated by Wang et al. 2020a).
During the installation, significant soil backflow over the back of the spudcan was observed, which consequently generated a tensile capacity when the spudcan footing was extracted from the soil. Suction developed since the drainage path to the underside of the spudcan was obstructed by the backflow soil (Cassidy et al., 2004a). Because the clay was remoulded during penetration with a reduction in the undrained shear strength, the tension capacity during extraction in the disturbed clay was lower than that of the vertical bearing capacity.

The ratio of the vertical tensile capacity to the compressive capacity, $\chi$, has been measured as 0.6 by Zhang et al. (2014a) and 0.65 by Hossain and Dong (2014) for spudcans in clay only. In the present experiments of clay overlying sand, the values of $\chi$ were 0.4 and 0.3 at $T/D = 0.38$ and $T/D = 0.27$, respectively. The value of $\chi$ decreased as the footing neared the underlying sand layer (decreasing $T/D$), since the underlying sand layer did not contribute to the tensile capacity to the same extent as the compressive capacity.

Fig. 8 shows the derived bearing capacity factor ($V/A_{su}$) as a function of the normalized penetration depth $w/D$. The mechanism-based design approach proposed by Hossain and Randolph (2009) for predicting the bearing capacity of spudcans in a single layer is also shown for comparison. The bearing capacity factors exhibit similar trends to the prediction curve until the penetration depth is approximately $1.0D$. This depth is at $0.4D$ above the clay-sand interface, indicating an influence zone within which the spudcan starts to ‘sense’ the underlying stronger sand layer and soil squeezing occurs. The depth for the onset of soil backflow is estimated by the algebraic expression given by Hossain and Randolph (2009), suggesting that a fully localized flow-around failure is developed. The region of the mobilised soil beneath the spudcan is $0.4D$ from the particle image velocimetry analysis in centrifuge tests reported by Hossain et al. (2005). This is consistent with the present observation of the depth for the onset of soil squeezing.

This deforming region of $0.4D$ is also supported by the velocity field for a deeply buried circular plate foundation in cohesive soil obtained from the limit analysis (Martin and Randolph, 2001). It is believed that the penetrating spudcan mobilises a similar extent of soil as the plate foundation, provided that the spudcan geometry fits within the rigid wedges of soil above and below the plate (Zhang et al. 2012).
A new prediction expression has recently been proposed to evaluate the vertical bearing capacity of spudcan foundations in clay overlying sand (Wang et al., 2021). This expression is based on the evolving soil failure mechanisms and is recommended to be used in the macro-element model as the hardening law, which relates the evolution of yield surface size with plastic vertical displacement, \( w_p \). This expression uses the conventional definition of penetration resistance for preloading and therefore must be transformed to the vertical bearing capacity provided by the soil, \( V_0 \), for subsequent use in determining the size of the yield surface. For a spudcan embedded beyond the onset of soil backflow, \( V_0 \) can be calculated according to

\[
V_o = q_u A - \gamma_s V_i
\]  
(2)

where \( q_u \) is the spudcan penetration resistance, \( A = \pi D^2 / 4 \) is the largest cross-sectional area of the spudcan, and \( V_i \) is the volume of the embedded spudcan. \( \gamma_s V_i \) represents the buoyancy force experienced by the spudcan. Between the depth at which the squeezing prevails and the clay-sand interface, the penetration resistance \( q_u \) is expressed as

\[
q_u = q_t + (q_u - q_b) \left[ 1 - (T_f / T_i)^n \right]^{1/m}
\]  
(3)

where \( q_t \) is the penetration resistance of spudcans in single-layer clay at a depth of \( T_i \) above the clay-sand interface, \( q_b \) is the penetration resistance at the sand surface, \( T_f \) is the depth of the deep flow-around mechanism, suggested as 0.4\( D \) based on experimental results in this study, and \( m = 1.3 \) and \( n = 2.2 \) are constants.

The value of \( q_t \) can be calculated as

\[
q_t = N_c s_c + \frac{\gamma_s V_i}{A}
\]  
(4)

where the deep bearing capacity factor \( N_c \) is given by Hossain and Randolph (2009),

\[
N_c = 10 \left( 1 + 0.05 \frac{w_p}{D} \right) \leq 11.3
\]  
(5)

where \( w_p = H_c - T_i \) is adopted.

The value of \( q_b \) can be estimated as

\[
q_b = \frac{1}{2} \gamma_s D N_c + p \gamma s q_d + \frac{\gamma_s V_i}{A}
\]  
(6)
where $N_{\gamma}$ and $N_{q}$ are dimensionless bearing capacity factors calculated for the axisymmetric case, $p_a$ is the effective overburden pressure, and $d_q$ is the depth factor. $N_{\gamma}$ for a conical footing on homogeneous sand given by Cassidy and Houlsby (2002) can be used, while $N_{q}$ calculated by Martin (2003) can be adopted.

Yield surface

**Correction to swipe tests**

The measured load paths in the swipe tests were corrected to account for soil elasticity and rig stiffness (Gottardi et al. 1999), in order to provide the loci of loads on the yield surface. Since the loading rig was not infinitely stiff and the elastic stiffness of the soil was not infinitely large, additional plastic penetration occurred to compensate for the decrease in the elastic penetration (because of the reduction in vertical load), maintaining the applied condition of $\Delta \omega = 0$ during the swipe test. A further adjustment was made to the measured load to consider the effect of changes in centrifugal accelerations. When rotation was applied at the RP, part of the footing moved into the higher gravity field, part to the lower gravity field, which subsequently generated a geo-force at the deviated mass centre away from the centreline. This geo-force was a function of the rotation $\theta$ and could be transferred to the $VHM$ load components at the RP. The load correction for the geo-force effect was then applied to all the swipe tests with a rotation. Fig. 9 shows the original and corrected results for test SW9, normalized by $V_0$. The rig flexibility and soil elasticity account for some of the 5% change in $V_0$; however, this is balanced by the opposite effect of the geo-force.

**Yield surface expression**

The yield surface expression proposed by Martin and Houlsby (2000) and Zhang et al. (2014a) for spudcans in single-layer clay is extended to fit the experimental data for spudcans in clay overlying sand. The closed-form expression is

$$f = \left[ \frac{H}{h_0 V_0} \right]^2 + \left[ \frac{M}{m_0 V_0} \right]^2 - 2 \frac{2 \varepsilon \Delta M / \Delta D}{h_0 m_0 V_0^2} - \frac{1}{\beta_1 \beta_2^2} \left[ \frac{(\beta_1 + \beta_2)^{\theta + \phi_1}}{(1 + \chi)^{\theta + \phi_1}} \right] \frac{1}{\phi} \frac{V}{V_0 + \chi} \left( \frac{1 - V}{V_0} \right)^2 = 0$$

(7)

where $h_0$ is the normalized peak horizontal load capacity, $m_0$ is the normalized peak moment capacity, $\varepsilon$ is the eccentricity of the elliptical section in the $HM$ plane, and $\chi$ is the ratio of the vertical tensile capacity to the compressive capacity (tensile capacity first introduced by Murff, 1994, but a different form of Vlahos 2004 is used in this paper). The fitting parameters
\( \beta_1 \) and \( \beta_2 \) define the curvature of the yield surface and the location of the peak (as introduced by Nova and Montrasio, 1991).

The best-fit parameters for the yield surface, obtained by performing the least-squares regression analysis at two embedment depths, are summarised in Table 4. To facilitate clear observation of the fit, a normalized non-vertical force is defined as

\[
\frac{Q}{V_0} = \sqrt{\left( \frac{H}{h_0 V_0} \right)^2 + \left( \frac{M/D}{m_0 V_0} \right)^2 - \frac{2 e H M / D}{h_0 m_0 V_0^2}}
\]

(8)

allowing equation (7) to be simplified as

\[
f = \left( \frac{Q}{V_0} \right)^2 - \left( \frac{\beta_1 + \beta_2 \chi}{1 + \chi} \right) \left( \frac{V}{V_0} \right)^{2 \beta_1} \left( 1 - \frac{V}{V_0} \right)^{2 \beta_2} = 0
\]

(9)

Fig. 10 shows the results of all the swipe tests performed at \( T/D = 0.38 \) and \( T/D = 0.27 \). By using equation (9), all the load paths of the swipe tests in the \( VHM \) loading space collapse into a ‘modified parabola’ in the normalized \( VQ \) plane, allowing all the swipe paths to be readily compared on one curve. It can be seen that equation (9) provides a close fit to the experimental data, with parameters given in Table 4. The value of \( \chi \) defines the left apex of the yield surface for the tensile vertical load. The curvature of the yield surface and the location of the peak are well captured using \( \beta_1 < \beta_2 < 1 \) consistently for all the embedment depths within the influence zone.

Fig. 11 shows the cross-sections in the \( HM \) plane through the normalized yield surface on the planes of a constant \( V/V_0 \). The data points of \( V/V_0 = 0.8, 0.7, 0.6, \) and \( 0.5 \) are compared with the corresponding two-dimensional slices of the fitted yield surface. It is further demonstrated that equation (7) can satisfactorily describe the swipe results. The yield surfaces in the \( HM \) loading space are not symmetrical with respect to the \( H \) and \( M \) axes. Equation (7) allows a rotated ellipse to be fitted with the inclusion of the eccentricity \( e \). The eccentricity of the ellipse is positive as the spudcan remains fully embedded in the upper clay layer, which is consistent with the experimental observations for spudcans in a single-layer clay (Martin and Houlsby, 2000; Cassidy et al., 2004a; Zhang et al., 2014a).
Yield surfaces in the normalized $VH$ and $VM$ loading spaces at $T/D = 0.38$ and $T/D = 0.27$ are compared in Fig. 12, respectively. As the spudcan nears the underlying sand layer, the absolute response in terms of $V$ and $M$ increases but remains unchanged in terms of $H$. The normalized yield surface therefore shrinks along the horizontal axis but remains unchanged along the moment axis. Although swipe tests were not performed in the tensile loading region, the load paths in the present study cover most of the positive vertical load quadrants, considering that the load combinations for spudcans are mainly in the region of $V > 0$ in practice. The tensile capacity is considered herein for the integrated definition of the yield surface in the three-dimensional $VHM$ loading space.

**Deriving yield surface parameters**

Fig. 13 shows the variation in the normalized size of the yield surface in the horizontal ($h_0$) and moment ($m_0$) directions with embedment. For a spudcan in clay overlying sand, the results from the present centrifuge tests are compared with the finite element (FE) analyses presented by Wang et al. (2020b). Within $0.4D$ above the clay-sand interface, $h_0$ decreases as the spudcan nears the underlying sand layer, while the change in $m_0$ is minimal. These trends are effectively captured by the wished-in-place numerical analyses, although the magnitude of $h_0$ is higher than the experimental data. The difference is due to the remoulded soil concentrated around the spudcan during the initial penetration process, which is not considered with the wished-in-place assumption. In contrast, the installation has less influence on the moment capacity because a larger mobilized rotational failure mechanism is exhibited in the soil for the moment capacity. This leads to good agreement for the value of $m_0$ between the FE results and centrifuge results.

The published solutions for spudcan foundations in single-layer clay are also shown in Fig. 13, which are derived from (i) $1g$ tests by Martin and Houlsby (2000) on heavily overconsolidated kaolin clay, (ii) centrifuge tests by Zhang et al. (2014a) on normally consolidated kaolin clay, (iii) FE analyses by Zhang et al. (2014b) taking into account the spudcan installation effect, and (iv) FE analyses by Zhang et al. (2011) for a wished-in-place spudcan. The differences among those solutions stem from the effects of soil backflow and the installation process, which has been discussed by Zhang et al. (2014a) and Zhang et al. (2014b). By comparing the results for the layered soil deposits and single-layer clay, it is suggested that the values of $h_0$ and $m_0$ for spudcan foundations in clay overlying sand can be scaled from the solutions for the single-layer clay. The scaling factors proposed by Wang et
al. (2020b) are recommended for the macro-element model to derive appropriate parameters for the yield surface. For $T/D \leq 0.4$, the effects of the underlying sand layer on the maximum horizontal ($h_0$) and moment ($m_0$) dimensions of the yield surface can be calculated as a function of $T/D$

\[
\frac{h_0}{h_{0,\text{clay}}} = \frac{1}{1 + 16.44 \left(0.4 - \frac{T}{D}\right)^{3.14}} \tag{10}
\]

\[
\frac{m_0}{m_{0,\text{clay}}} = 1.0 \tag{11}
\]

where $h_{0,\text{clay}}$ and $m_{0,\text{clay}}$ are the dimensions of the yield surface in single-layer clay. These values can be determined from the solutions of Zhang et al. (2014b), as indicated in Fig. 13.

The eccentricity of the rotated ellipse in the $HM$ plane is

\[
\frac{e}{e_{\text{clay}, T/D=0.4}} = \frac{1}{1 + 37.89 \left(0.4 - \frac{T}{D}\right)^{3.42}} \tag{12}
\]

where $e_{\text{clay}, T/D=0.4}$ is the value at $T/D = 0.4$, which can be estimated from the parameter $e$ for the spudcan foundation in single-layer clay at an embedment depth of $w = H - 0.4D$

The value of $\chi$ can be determined by assuming that the effect of the underlying sand layer on the tensile capacity is negligible; thus, $\chi$ is the ratio of the original tensile capacity to the increased compressive capacity due to soil squeezing. For simplicity, the original tensile capacity is taken as 60% of the compressive capacity for spudcan foundations in single-layer clay. $\beta_1 = 0.37$ and $\beta_2 = 0.87$ are held constant for all embedment depths within $T/D \leq 0.4$.

**Elastic behaviour**

Inside the yield surface the relationship between the load increments ($\delta V$, $\delta H$, $\delta M$) and the corresponding elastic displacements ($\delta w_e$, $\delta u_e$, $\delta \theta_e$) is

\[
\begin{bmatrix}
\delta V \\
\delta H \\
\delta M / D
\end{bmatrix} = G_{\text{clay}} D \begin{bmatrix}
4 K_v & 0 & 0 \\
0 & 4 K_h & 2 K_c \\
0 & 2 K_c & K_m
\end{bmatrix} \begin{bmatrix}
\delta w_e \\
\delta u_e \\
D \delta \theta_e
\end{bmatrix} \tag{13}
\]

where $G_{\text{clay}}$ is the shear modulus of the clay and $K_v$, $K_h$, $K_m$, and $K_c$ are dimensionless elastic stiffness coefficients derived by FE analysis. Wang et al. (2020a) published a comprehensive series of cases for a circular footing on a soft clay over stiff sand, covering a wide range of foundation embedment depths, the shear modulus ratios between the layers, and the clay
layer thickness below the footing. The shear modulus of clay, $G_{\text{clay}}$, can generally be expressed as

$$G_{\text{clay}} = I_r s_a$$  \hspace{1cm} (14)

where $I_r$ is the rigidity index. The shear modulus of sand, $G_{\text{sand}}$, is correlated to the stress level, which can be evaluated by (Cassidy et al., 2004b)

$$G_{\text{sand}} = g_s p_s \left( \frac{V}{A p_s} \right)^{0.5}$$ \hspace{1cm} (15)

where $V$ is the representative vertical load on the foundation, $A$ is the foundation area, $p_s$ is atmospheric pressure, and $g_s$ is a dimensionless constant. A typical value of $g_s$ is approximately 400 for medium dense sand (Houlsby and Cassidy, 2002). The shear modulus ratio $G_{\text{sand}}/G_{\text{clay}}$ can, therefore, be calculated and used in choosing the stiffness coefficients from the results of Wang et al. (2020a).

Table 5 presents the elastic stiffness coefficients relevant to the investigated embedment depths. The unloading-reloading loops performed in the vertical penetration tests provided information about the vertical elastic stiffness. By considering the dimensionless stiffness coefficients given in Table 5 and equation (14), an average value $I_r$ of 30 was found to fit the experimental data well.

**Flow rule**

When the load combination reaches the yield surface, the surface expands, and a flow rule is required to determine the ratios of plastic displacements. An associated flow rule has been found to accurately model the flow behaviour in the $HM$ plane (Martin and Houlsby, 2001; Houlsby and Cassidy, 2002; Zhang et al., 2014a), while the non-associated flow was assumed for the $VQ$ plane. Thus, a plastic potential of a similar form to the yield surface is proposed

$$g = \left( \frac{H}{h_v V_o} \right)^2 + \left( \frac{M / D}{m_v V_o} \right)^2 - \frac{2 e H M / D}{h_v m_v V_o^2} - \alpha_v^2 \left[ \left( \frac{V}{V_o} \right)^{\beta_3 + \beta_4} \frac{1}{(1 + \chi) (\beta_3 + \beta_4)} \left( \frac{V}{V_o} + \chi \right)^{\beta_3} \left( 1 - \frac{V}{V_o} \right)^{2 \beta_4} \right] = 0$$ \hspace{1cm} (16)

where $V_o$ is a dummy parameter defining the intersection of the plastic potential surface that passes through the current load point with the vertical load axis, $\alpha_v$ is an association parameter ($\alpha_v = 1.0$ representing associated flow), and $\beta_3$ and $\beta_4$ are the modified curvature parameters.
For plastic displacements in the $HM$ plane, associated flow was tested by comparing the measured plastic displacement ratios $\delta u_p / D \delta \theta_p$ during the swipe tests with the theoretical predictions of associated flow. The comparison is shown in Fig. 14 for $T/D = 0.27$. Based on plasticity theory, the ratio of the plastic horizontal displacement to the rotation is

$$\frac{\delta u_p}{D \delta \theta_p} = \frac{\partial g / \partial H}{\partial g / \partial (M / D)} - \frac{(m_h/h_o) (HD/M) - e}{(h_u/m_o) - e (HD/M)}$$  \hspace{1cm} (17)$$

where $\delta u_p$ and $\delta \theta_p$ are increments of the plastic horizontal and rotational displacement, respectively. Equation (17) represents the associated flow assumption in the $HM$ plane as $\delta u_p / \delta \theta_p = (\partial f / \partial H) / (\partial f / \partial M)$. In general, good agreement is found between the measured data and the predicted plastic displacement ratios by assuming associated flow, though slight underestimation of $|\delta u_p / D \delta \theta_p|$ at a given $HD/M$ ratio can be seen for the load state in the ($-H, M$) quadrant. However, these load states are rarely encountered in offshore practice. $H$ and $M$ are generally of the same sign as the moment acting on the foundation is typically generated by the horizontal load at the elevated height on the superstructure. Thus, it is reasonable to assume an associated flow in the $HM$ plane for spudcans in clay overlying sand.

Tests that expand the yield surface in a predefined manner are required to provide information on the degree of non-association in the $VQ$ plane, including radial displacement tests and constant vertical load tests. None of these tests were conducted in this study. However, since the spudcan foundation remained embedded in the clay layer, a flow behaviour similar to that previously measured for a spudcan in single-layer clay is assumed.

Zhang et al. (2014a) modelled such behaviours in the centrifuge and proposed appropriate parameters for the plastic potential. $\alpha_v = 1.1$ and $\beta_v = 0.45$ were held the same as those by Zhang et al. (2014a). However, the value of $\beta_3$ is decreased to 0.20 to match the condition of $\beta_3 < \beta_2$. This value is chosen on the basis that the ratio of the plastic potential curvature parameters, $\beta_3 / \beta_4$, is equal to the ratio of the yield surface curvature parameters, $\beta_4 / \beta_5$. This ensures that the position of the ‘parallel point’ on the yield surface coincides with the peak of the yield surface, as suggested by the experimental observations. The ‘parallel point’ is defined as the point on the yield surface at which the load state does not change despite continuing deviatoric displacements (Tan, 1990; Houlsby and Cassidy, 2002). Therefore, the
parameters were summarised as $\alpha_1 = 1.1$, $\beta_1 = 0.20$, and $\beta_4 = 0.45$, which were found to predict the experimental data well, as illustrated in the following retrospective simulations.

**RETROSPECTIVE ANALYSES**

The macro-element model has been implemented as a Fortran 90 program. Numerical macro-element simulations for representative centrifuge tests have been performed to investigate the capability of the model. The input of the model was taken as the experimentally prescribed displacements, and the loads on the spudcan were calculated as the output.

**Pure vertical loading**

Fig. 15 shows the retrospective simulations of the vertical loading tests. The results are presented in terms of the conventional definition of the penetration resistance for preloading, $q_u$, as a function of the normalized penetration depth, $w/D$. The substantially increased penetration resistance associated with soil squeezing predicted by the model is in good agreement with the experimental observations. The depth for the onset of squeezing is also well simulated. The prediction by the design guideline ISO (2016) is included for comparison in Fig. 15, which reaches the maximum bearing pressure experimentally experienced by the footing at a deeper penetration depth. Furthermore, the penetration resistance starts to increase sharply when the spudcan is much closer to the sand layer, which is not the case based on the experimental results.

**Swipe tests**

Fig. 16 and Fig. 17 show the results of the macro-element simulations of swipe tests performed in the $(+H, +M)$ quadrant at $T/D = 0.38$ (SW1 to SW4) and $T/D = 0.27$ (SW7 to SW10), respectively. The simulations followed the experiments by firstly prescribing a purely vertical displacement before it was held constant and the horizontal displacement and rotation were then applied as shown in Table 3. Therefore, the swipe phase of the tests commenced from $V/V_o = 1$ and tracked the yield surface. The swipe tests started at the same loads of $V_0$ as those experimentally experienced by the spudcan. In general, there is good agreement between the predicted load paths and the experimental data. The model satisfactorily predicts the position of the ‘parallel point’ with the assumed parameters for the plastic potential. The model tracks a yield surface slightly outside the experimental load path in the $VH$ plane, while it slightly underpredicts the surface in the $VM$ plane. Although the
model predicts the load-displacement curve for the rotational response well, a stiffer horizontal response is simulated than that in the experiments. While the ability of the model was demonstrated through the retrospective analyses, it should be noted that the model parameters pertain to experimental conditions where the spudcan penetrated well beyond the onset of backflow and significant soil backflow was observed. This is the typical scenario for deep spudcan penetration in offshore soft clay sites (Hossain et al., 2005). However, an open cavity may be presented above the footing in stiff clay and without backflow the size parameters of the yield surface are lower (Martin and Houlsby, 2001). Other effects include decreased elastic stiffnesses and reduced tension capacity due to loss of soil contact underneath the footing. Thus, model parameters need to be re-evaluated for the spudcans embedded shallower than the onset of backflow in the upper clay layer. In addition, the macro-element model has only been tested by monotonic loading events, therefore it cannot be extended to the modelling of cyclic behaviour that involves hysteretic loops and gradual degradation of stiffness within the yield surface.

CONCLUSIONS

This paper presents a plasticity-based macro-element model describing the load-displacement behaviour of a spudcan foundation in clay overlying sand when subjected to combined VHM loading. The results from centrifuge tests designed to explore the effect of the underlying sand layer on the foundation behaviour within the influence zone in the upper clay layer have been discussed. Experimental, theoretical and numerical developments have been combined to derive the components of the model. The predictive capabilities of the macro-element model have been verified by retrospective simulations of the experiments. The macro-element model developed in this paper can be incorporated into structural analyses of mobile jack-up rigs (e.g., Williams et al., 1998; Martin and Houlsby, 1999; Houlsby and Cassidy, 2002; Bienen and Cassidy, 2009), allowing for more accurate modelling of the offshore structures when deployed in clay overlying sand deposits.

ACKNOWLEDGEMENTS

The first author acknowledges the support of an Australian Government Research Training Program (RTP) Scholarship.
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A$</td>
<td>largest cross-sectional area of spudcan</td>
</tr>
<tr>
<td>$c_v$</td>
<td>coefficient of consolidation</td>
</tr>
<tr>
<td>$D$</td>
<td>spudcan diameter</td>
</tr>
<tr>
<td>$D_{T\text{-bar}}$</td>
<td>T-bar diameter</td>
</tr>
<tr>
<td>$d_4$</td>
<td>depth factor</td>
</tr>
<tr>
<td>$e$</td>
<td>eccentricity of elliptical section in $HM$ plane</td>
</tr>
<tr>
<td>$e_{\text{clay, } T/D=0.4}$</td>
<td>eccentricity of yield surface in single-layer clay at $T/D = 0.4$</td>
</tr>
<tr>
<td>$f$</td>
<td>yield surface function</td>
</tr>
<tr>
<td>$G_{\text{clay}}$</td>
<td>shear modulus of clay</td>
</tr>
<tr>
<td>$G_{\text{sand}}$</td>
<td>shear modulus of sand</td>
</tr>
<tr>
<td>$g$</td>
<td>plastic potential function</td>
</tr>
<tr>
<td>$g_g$</td>
<td>non-dimensional shear modulus factor</td>
</tr>
<tr>
<td>$H$</td>
<td>horizontal load</td>
</tr>
<tr>
<td>$H_c$</td>
<td>clay thickness</td>
</tr>
<tr>
<td>$h_0$</td>
<td>normalized peak horizontal load capacity</td>
</tr>
<tr>
<td>$h_{0, \text{clay}}$</td>
<td>horizontal dimension of yield surface in single-layer clay</td>
</tr>
<tr>
<td>$I_D$</td>
<td>relative density</td>
</tr>
<tr>
<td>$I_r$</td>
<td>rigidity index</td>
</tr>
<tr>
<td>$K_v, K_h, K_m, K_c$</td>
<td>dimensionless elastic stiffness coefficients</td>
</tr>
<tr>
<td>$k$</td>
<td>shear strength gradient</td>
</tr>
<tr>
<td>$M$</td>
<td>moment load</td>
</tr>
<tr>
<td>$m$</td>
<td>fitting parameter in equation for hardening law</td>
</tr>
<tr>
<td>$m_0$</td>
<td>normalized peak moment load capacity</td>
</tr>
<tr>
<td>$m_{0, \text{clay}}$</td>
<td>moment dimension of yield surface in single-layer clay</td>
</tr>
<tr>
<td>$N_{c}, N_{T}, N_{q}$</td>
<td>bearing capacity factors</td>
</tr>
<tr>
<td>$N_{T\text{-bar}}$</td>
<td>bearing factor of T-bar</td>
</tr>
<tr>
<td>$n$</td>
<td>fitting parameter in equation for hardening law</td>
</tr>
<tr>
<td>$p_\cdot$</td>
<td>effective overburden pressure</td>
</tr>
<tr>
<td>$p_a$</td>
<td>atmospheric pressure</td>
</tr>
<tr>
<td>$Q$</td>
<td>non-vertical force</td>
</tr>
<tr>
<td>$q_b$</td>
<td>penetration resistance at sand surface</td>
</tr>
<tr>
<td>$q_t$</td>
<td>penetration resistance at the depth of $T_f$ above clay-sand interface</td>
</tr>
<tr>
<td>$q_{T\text{-bar}}$</td>
<td>penetration resistance of T-bar</td>
</tr>
<tr>
<td>$q_u$</td>
<td>spudcan penetration resistance</td>
</tr>
<tr>
<td>$s_u$</td>
<td>undrained shear strength</td>
</tr>
<tr>
<td>$s_{um}$</td>
<td>shear strength at mudline</td>
</tr>
<tr>
<td>$T$</td>
<td>thickness of clay layer below spudcan</td>
</tr>
<tr>
<td>$T_f$</td>
<td>depth of failure mechanism beneath spudcan</td>
</tr>
<tr>
<td>$u$</td>
<td>horizontal displacement</td>
</tr>
<tr>
<td>$u_e$</td>
<td>elastic horizontal displacement</td>
</tr>
<tr>
<td>$u_p$</td>
<td>plastic horizontal displacement</td>
</tr>
<tr>
<td>$\dot{u}$</td>
<td>horizontal displacement rate</td>
</tr>
<tr>
<td>$V$</td>
<td>vertical load</td>
</tr>
<tr>
<td>$V_0$</td>
<td>maximum vertical load experienced by spudcan</td>
</tr>
<tr>
<td>$V_\cdot$</td>
<td>maximum vertical load for current plastic potential shape</td>
</tr>
</tbody>
</table>
\( V_t \): volume of embedded spudcan
\( w \): vertical displacement
\( w_e \): elastic vertical displacement
\( w_p \): plastic vertical displacement
\( \dot{w} \): vertical displacement rate
\( z \): soil depth
\( \alpha_v \): association parameter
\( \beta_1, \beta_2 \): curvature factor exponents in equation for yield surface
\( \beta_3, \beta_4 \): curvature factor exponents in equation for plastic potential
\( \gamma_c' \): submerged unit weight of clay
\( \gamma_s' \): submerged unit weight of sand
\( \theta \): rotational displacement
\( \theta_e \): elastic rotational displacement
\( \theta_p \): plastic rotational displacement
\( \dot{\theta} \): rotational displacement rate
\( \chi \): ratio of vertical tensile over compressive capacity

REFERENCES


**TABLE CAPTIONS**

Table 1. Soil properties of superfine silica sand (after Chow et al., 2019).
Table 2. Kaolin clay properties (after Stewart, 1992; Acosta-Martinez and Gourvenec, 2006).
Table 3. Details of the swipe tests (model scale).
Table 4. Summary of the best-fit yield surface parameters.
Table 5. Representative elastic stiffness coefficients (after Wang et al., 2020a).
Table 1. Soil properties of superfine silica sand (after Chow et al., 2019).

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity, $G_s$</td>
<td>2.67</td>
</tr>
<tr>
<td>Average effective particle size, $d_{50}$: mm</td>
<td>0.18</td>
</tr>
<tr>
<td>Maximum void ratio, $e_{\text{max}}$</td>
<td>0.784</td>
</tr>
<tr>
<td>Minimum void ratio, $e_{\text{min}}$</td>
<td>0.505</td>
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<tr>
<td>Critical state friction angle, $\phi_{cv}$: degrees</td>
<td>31</td>
</tr>
</tbody>
</table>

Table 2. Kaolin clay properties (after Stewart, 1992; Acosta-Martinez and Gourvenec, 2006).

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
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<tbody>
<tr>
<td>Liquid limit, LL: %</td>
<td>61</td>
</tr>
<tr>
<td>Plastic limit, PL: %</td>
<td>27</td>
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<tr>
<td>Plasticity index, $I_p$: %</td>
<td>34</td>
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<tr>
<td>Specific gravity, $G_s$</td>
<td>2.6</td>
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<tr>
<td>Angle of internal friction, $\phi^\prime$: degrees</td>
<td>23.5</td>
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<tr>
<td>Critical state frictional constant, $M$</td>
<td>0.92</td>
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<tr>
<td>Slope of the normal consolidation line, $\lambda$</td>
<td>0.205</td>
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<tr>
<td>Slope of the swelling line, $\kappa$</td>
<td>0.044</td>
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<tr>
<td>Coefficient of consolidation, $c_v$: m$^2$/year</td>
<td>2</td>
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Table 3. Details of the swipe tests (model scale).

<table>
<thead>
<tr>
<th>w/D</th>
<th>T/D</th>
<th>Test name</th>
<th>$V_0$ (N)</th>
<th>$u/D\theta$</th>
<th>Swipe displacements</th>
<th>Velocities</th>
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<tr>
<td></td>
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<td>$u$ (mm)</td>
<td>$\theta$ (°)</td>
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<tr>
<td>1.04</td>
<td>0.38</td>
<td>SW1</td>
<td>907.1</td>
<td>$\infty$</td>
<td>12</td>
<td>0</td>
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<tr>
<td></td>
<td></td>
<td>SW2</td>
<td>966.0</td>
<td>1.43</td>
<td>12</td>
<td>8</td>
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<tr>
<td></td>
<td></td>
<td>SW3</td>
<td>957.0</td>
<td>0.95</td>
<td>9</td>
<td>9</td>
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<tr>
<td></td>
<td></td>
<td>SW4</td>
<td>932.6</td>
<td>-0.095</td>
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<td>9</td>
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<tr>
<td></td>
<td></td>
<td>SW5</td>
<td>927.2</td>
<td>-0.57</td>
<td>-5.4</td>
<td>9</td>
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<td></td>
<td></td>
<td>SW6</td>
<td>862.9</td>
<td>-1.15</td>
<td>-9</td>
<td>7.5</td>
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<tr>
<td>1.15</td>
<td>0.27</td>
<td>SW7</td>
<td>1160.2</td>
<td>$\infty$</td>
<td>12</td>
<td>0</td>
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<tr>
<td></td>
<td></td>
<td>SW8</td>
<td>1262.4</td>
<td>1.43</td>
<td>12</td>
<td>8</td>
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<td></td>
<td></td>
<td>SW9</td>
<td>1307.9</td>
<td>0.95</td>
<td>9</td>
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<td></td>
<td></td>
<td>SW10</td>
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<td>-0.9</td>
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<td></td>
<td></td>
<td>SW11</td>
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<td>-5.4</td>
<td>9</td>
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<tr>
<td></td>
<td></td>
<td>SW12</td>
<td>1257.0</td>
<td>-1.15</td>
<td>-9</td>
<td>7.5</td>
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Table 4. Summary of the best-fit yield surface parameters.

<table>
<thead>
<tr>
<th>w/D</th>
<th>T/D</th>
<th>$h_0$</th>
<th>$m_0$</th>
<th>$e$</th>
<th>$\chi$</th>
<th>$\beta_1$</th>
<th>$\beta_2$</th>
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</thead>
<tbody>
<tr>
<td>1.04</td>
<td>0.38</td>
<td>0.232</td>
<td>0.135</td>
<td>0.462</td>
<td>0.4</td>
<td>0.37</td>
<td>0.87</td>
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<tr>
<td>1.15</td>
<td>0.27</td>
<td>0.206</td>
<td>0.137</td>
<td>0.513</td>
<td>0.3</td>
<td>0.37</td>
<td>0.87</td>
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</tbody>
</table>
Table 5. Representative elastic stiffness coefficients (after Wang et al., 2020a).

<table>
<thead>
<tr>
<th>w/D</th>
<th>T/D</th>
<th>K_v</th>
<th>K_h</th>
<th>K_m</th>
<th>K_c</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.04</td>
<td>0.38</td>
<td>25.41</td>
<td>12.85</td>
<td>13.11</td>
<td>0.88</td>
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<tr>
<td>1.15</td>
<td>0.27</td>
<td>27.88</td>
<td>13.83</td>
<td>15.09</td>
<td>0.87</td>
</tr>
</tbody>
</table>

FIGURE CAPTIONS

Fig. 1. Schematic diagram of the drum centrifuge and experimental equipment
Fig. 2. Sign convention of loads and displacements
Fig. 3. General view of the loading apparatus
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Fig. 16. Retrospective macro-element simulation of the swipe tests at T/D = 0.38: line colour represents physical experiment or numerical simulation and line type the test name
Fig. 17. Retrospective macro-element simulation of the swipe tests at T/D = 0.27: line colour represents physical experiment or numerical simulation and line type the test name
Fig. 1
Fig. 2
Fig. 3
Fig. 4
Fig. 5
Fig. 6

Swipe tests from $V_0$
Fig. 7
Bearing capacity factor, $N_c = V/As_u$

- Depth of back-flow
- Vertical loading tests & Initial penetration of swipes
- Prediction of single-layer clay (Hossain & Randolph, 2009)

Fig. 8
Equation (9)

Fig. 10(a)

Equation (9)

Fig. 10(b)
**Fig. 13(a)**

- **Solid lines:** Clay-over-sand
  - **Red** Centrifuge tests (This study)
  - **Orange** FE: wished-in-place (Wang et al., 2020b)

- **Dashed lines:** Single-layer clay
  - **Gray** 1g tests (Martin & Housby, 2000)
  - **Gray Square** Centrifuge tests (Zhang et al., 2014a)
  - **Gray Triangle** FE: considering installation effects (Zhang et al., 2014b)
  - **Gray Circle** FE: wished-in-place (Zhang et al., 2011)

---

**Fig. 13(b)**

- **Solid lines:** Clay-over-sand
  - **Red** Centrifuge tests (This study)
  - **Orange** FE: wished-in-place (Wang et al., 2020b)

- **Dashed lines:** Single-layer clay
  - **Gray** 1g tests (Martin & Housby, 2000)
  - **Gray Square** Centrifuge tests (Zhang et al., 2014a)
  - **Gray Triangle** FE: considering installation effects (Zhang et al., 2014b)
  - **Gray Circle** FE: wished-in-place (Zhang et al., 2011)
Fig. 14