MODELLING THE EFFECTS OF PAUSES DURING SPUDCAN PENETRATION ON FURTHER INSTALLATION BEHAVIOUR

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ABSTRACT
The installation of jack-up platforms may result in a discontinuous process due to either expected or unexpected delays experienced while penetrating. For this reason, it is important to correctly estimate not only the response of the soil during penetration but also the effect that a period of consolidation of variable length can have on further penetration. Modelling this problem numerically would allow a wider parametric study to be conducted in an economic fashion. However, it requires a large deformation approach, where coupled pore fluid pressure-effective stress analyses are able to describe the buildup of excess pore pressure during penetration as well as the relative dissipation during a stage of consolidation, with relative increases in the strength of the soil in the vicinity of the spudcan. For this reason, an advanced hypoplastic numerical model within the framework of the critical state soil mechanics for structured clays was implemented. Additionally, by implementing the structure within the model, the soil degradation due to shear strains can be described, starting the analysis with an intact shear strength that is gradually degraded as the spudcan advances. The response after consolidation is compared with results from experimental tests in a centrifuge environment at enhanced gravity, where a spudcan model equipped with a pore pressure transducer was penetrated. The results show that the effects of a period of consolidation must be taken into account because, depending on its length, it could lead to a significant increase in the bearing capacity on re-penetration due to the stiffening and strengthening of the soil, thereby offering insight into a potential shallower and yet reliable installation depth.

KEY WORDS: Jack-up platform; Spudcan penetration; Consolidation; Bearing capacity; Hypoplasticity; Numerical modelling; Centrifuge.

INTRODUCTION

The prediction of the spudcan load-penetration curve prior to jack-up installation is a common procedure in offshore engineering. Depending on the installation site, drained or undrained conditions are usually adopted for prediction on sand to silty-sand or clay to silty-clay, respectively (ISO 2012[1]). Such predictions are then compared in real time with the actual, in-field load-penetration curve, allowing the operators to evaluate whether the installation is unfolding as planned or not. As schematically represented in Figure 1, the process of installation, and in particular the leg penetration, is usually discontinuous in time. This is normally due to the operational requirements of the cyclical pumping of water into ballast tanks to penetrate the footings and dumping the tanks to jack up the hull, which repeatedly interrupts the penetration. This can be considered a standard procedure as only a few jack-up units allow mobilizing the hull with filled tanks, although it usually occurs with short pauses, typically no longer than a few hours. However, unexpected events that may extend a pause, such as poor weather conditions or technical problems, should be taken into account when predicting the load-penetration curve.

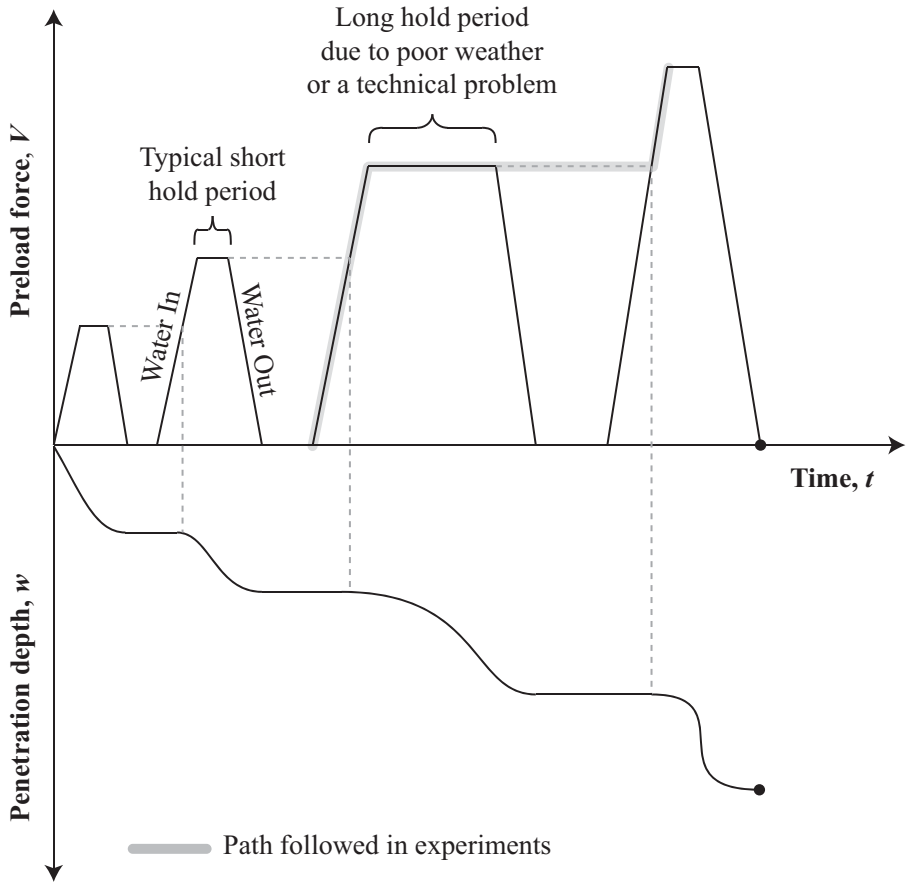
Previous experimental studies like those of Hossain et al. (2005)[2] and Zhang et al. (2014)[3] mainly focused on continuous installation, with little consideration of pauses in the literature. Barbosa-Cruz (2007)[4] and Bienen and Cassidy (2013)[5] conducted a limited number of centrifuge experiments on spudcan

penetration-consolidation-penetration cycles, showing that an increment in the bearing capacity is observed after the load is held constant for a certain period, with the magnitude of the event related to the length of the pause.

On the other hand, particular numerical simulation strategies are necessary as the traditional small strain finite element (FE) methods are unable to simulate the entire process due to the severe mesh distortion around the spudcan. Several large deformation finite element (LDFE) methods tried to overcome the issue through periodic regeneration of the Lagrangian mesh (Hossain et al. 2005[2], Zhang et al. 2014[3]) or allowing soil flowing within the Eulerian mesh (Tho et al. 2012[6]), but all were expressed in terms of continuous installation using total stress analyses. Wang et al. (2008)[7] instead proposed an LDFE approach in terms of the effective stress, which enables conducting coupled effective stress-pore fluid analyses and hence keeping track of excess pore pressure generation/dissipation.

This paper explores the effect of consolidation during spudcan installation, presenting a numerical study on the commercially available soil kaolin clay. A hypoplastic constitutive model for structured clays (Masin 2007[8]) was incorporated into the LDFE analysis based on the Remeshing and Interpolation Technique with Small Strains (RITSS) strategy (Hu and Randolph 1998[9]). A commercial package, Abaqus/Standard, was used to conduct the Lagrangian calculation and mesh regeneration. The results were compared and validated against drum centrifuge modelling test data by Bienen and Cassidy (2013)[5]. The LDFE simulation allows the prediction to be drawn in a more economic fashion and ultimately wider parametric studies to be conducted.

Figure 1: Schematic time history of preloading force and penetration during a discontinuous installation process.



HYPOPLASTICITY

Hypoplasticity is an approach to the non-linear constitutive modelling of geomaterials. In its general form developed by Gudehus (1996)[10], it may be written as

$$\dot{\boldsymbol{\sigma}} = f_s\left(\boldsymbol{L} : \dot{\boldsymbol{\varepsilon}} + f_d \boldsymbol{N}\left\|\dot{\boldsymbol{\varepsilon}}\right\|\right) \tag{1}$$

where $\dot{\boldsymbol{\sigma}}$ and $\dot{\boldsymbol{\varepsilon}}$ represent the objective (Zaremba-Jaumann) stress rate and the Euler stretching tensor, respectively, $\boldsymbol{L}$ and $\boldsymbol{N}$ are the fourth- and second-order constitutive tensors, and f_s and f_d are two scalar factors. In hypoplasticity, the stiffness predicted by the model is controlled by tensor $\boldsymbol{L}$, whereas the strength (and asymptotic response in general) is governed by a combination of $\boldsymbol{L}$ and $\boldsymbol{N}$. Earlier hypoplastic models, such as those of Wolffersdorff (1996)[11] and Mašín (2005)[12], did not allow arbitrarily changing the $\boldsymbol{L}$ formulation as any modification of tensor $\boldsymbol{L}$ undesirably influenced the predicted asymptotic states. This hypoplasticity limitation was overcome by Mašín (2012)[13]. He developed an approach enabling the specification of the asymptotic state boundary surface independently of tensor $\boldsymbol{L}$ and demonstrated it by proposing a simple hypoplastic equivalent of the Modified Cam-clay model. Based on this approach, Mašín (2014)[14] developed an advanced hypoplastic model for clays, which is adopted in the present work.

The hypoplastic model requires five material parameters φ_c, N, λ^*, κ^* and υ. The parameters have the same physical interpretation as the parameters of the Modified Cam clay model, and they are thus easy to calibrate using standard laboratory experiment. The model parameters N and λ^* define the position and the slope of the isotropic normal compression line in the $\ln(1+e)$ vs $\ln(p/p_r)$ plane. The isotropic normal compression line is described using the equation

$$\ln(1+e) = N - \lambda^* \ln\left(\frac{p}{p_r}\right) \tag{2}$$

where p_r = 1 kPa is a reference stress. Parameter κ^* controls the slope of the isotropic unloading line in the same plane and also of the isotropic compression line of overconsolidated soil. φ_c is the critical state friction angle, with identical meaning to that in any other critical state soil mechanics-based model. Finally, parameter υ controls the shear modulus.

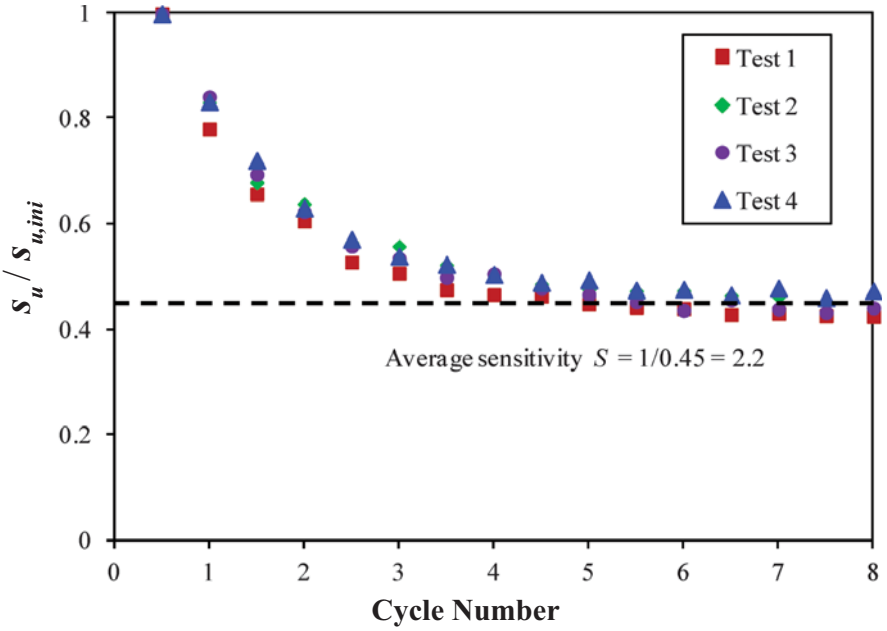
Apart from the stress, the most important state variable controlling the response of the model is the void ratio e. In the numerical simulation, this parameter can be directly initialised as a constant. Such an approach has, however, a shortcoming of increasing degree of overconsolidation with depth. In this case, the void ratio is initialised in an alternative manner, ensuring a constant overconsolidation ratio OCR with depth. The OCR in the hypoplastic model is defined as

$$OCR = \frac{p_e}{p} \tag{3}$$

where p_e is the Hvorslev equivalent pressure, that is, the mean effective stress at the isotropic normal compression line calculated using Equation (2) at the current void ratio. Note that K_0 normally consolidated soil does not have OCR = 1 as the oedometric normal compression line is below the isotropic normal compression line in the $\ln(1+e)$ vs $\ln(p/p_r)$ plane. To initialize the state for K_0 normally consolidated conditions, a value of OCR corresponding to these conditions was calculated from the model equations.

The experimental data using cyclic T-bar penetration in centrifuge tests (Stewart and Randolph 1994[15]) shown in Figure 2 indicate that kaolin clay samples show a remoulding degree, $s_u/s_{u,ini}$, where $s_{u,ini}$ is the initial, intact shear strength of the sample and s_u the remoulded value.

Figure 2: Episodes of cyclic T-bar penetration in a drum centrifuge showing structure degradation.



For this reason, the hypoplastic model was enhanced by the effects of the structure using a procedure developed in Mašín (2007)[8]. This enhancement requires three additional parameters, k, A and s_f, and the sensitivity s as a state variable. The normal compression line shown in Figure 3 then reads

$$\ln(1+e) = N - \lambda^* \ln\left(\frac{p}{p_r}\right) + \lambda^* \ln(s) \tag{4}$$

and the overconsolidation ratio is defined using $OCR = p_e/p = sp_e^*/p$, where p_e is the Hvorslev equivalent pressure on the isotropic normal compression line of the structured soil and p_e^* is the same equivalent pressure of the reconstituted soil.

Parameter A, which has relatively little effect on the results, was taken as 0.2. Parameter k was calibrated by means of a back-analysis using triaxial simulations (Figure 4) starting from different stress levels of isotropic normal consolidation ($p' = 50$, 100, 400 kPa, respectively) and centrifuge test data. The final value of the sensitivity s_f has been specified as 1, reflecting the results of the T-bar penetration tests.

The set of parameters adopted in the simulations is given in Table 1.

Table 1: Parameters of the hypoplastic model used in the simulations

φ_c	N	λ^*	κ^*	υ	k	A	s_f
23°	1.195	0.0811	0.019	0.2	0.2	0.2	1

Figure 3: Hypoplastic bilogarithmic compression law for structured clay.

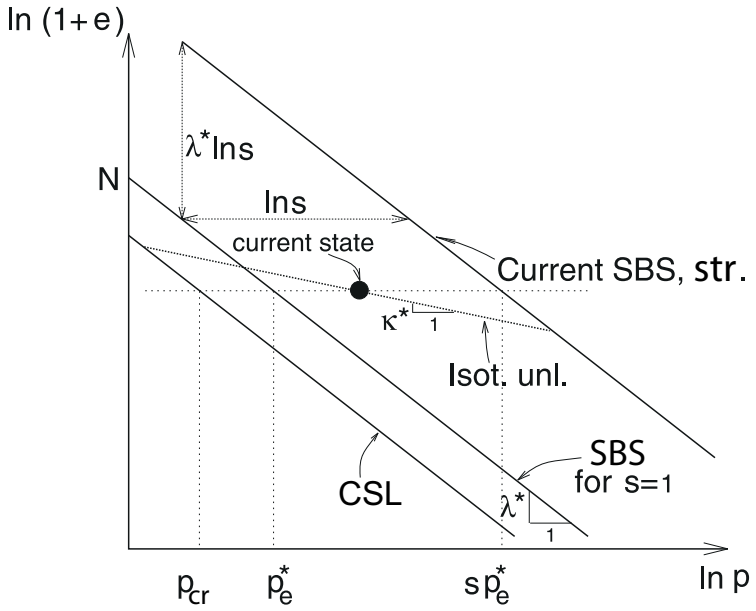
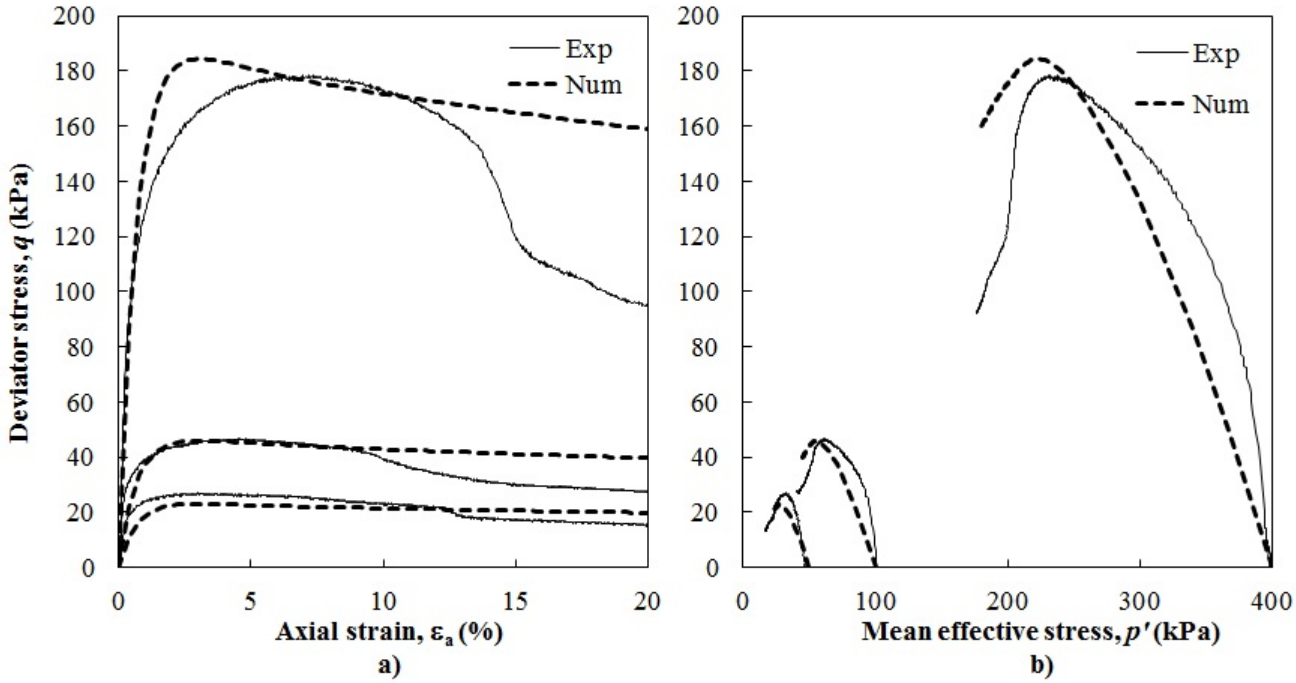


Figure 4: Numerical simulations of triaxial tests starting from isotropic normally consolidated conditions in the a) *q*-ε_a and b) *q*-*p'* planes.



LARGE DEFORMATION FINITE ELEMENT (LDFE) ANALYSES

Spudcan installation involves a large deformation of the soil around the footing, so the Abaqus-based RITSS was used to reproduce the penetration-consolidation-penetration problem. Furthermore, to capture the excess pore pressure built up in the penetration and relative dissipation during the consolidation, a coupled effective stress-pore fluid analysis was employed. The idea behind this strategy used to tackle large deformation problems is to divide the whole process into several small steps, such that the mesh is not excessively distorted. Once the current step is completed, the distorted soil is remeshed for the next step, whereas all the state variables at the integration points and the pore pressure at the element nodes are mapped from the old mesh to the new mesh. A detailed description of the implementation of the RITSS method in Abaqus can be found in Wang et al. (2013)[16].

Due to the axisymmetric geometry of the problem, 8-node biquadratic displacement, bilinear pore pressure, reduced integration elements (CAX8RP) were selected to simulate the experiments. The spudcan was modelled as a rigid body. The soil-spudcan interface was considered frictionless.

The hypoplastic constitutive model for clays is incorporated into the Lagrangian calculation in each step by coding a user subroutine UMAT. A UMAT for the hypoplastic model for clays with structure was available online by Gudehus et al. (2008)[17]. The shear strength is calculated as follows

$$s_u = \frac{OCR\ p'}{4} M \tag{5}$$

resulting in an undrained strength profile with a gradient of 1.66w kPa/m, where OCR = 1.33 refers to normally consolidated K_0 conditions, with $K_0 = 1 - \sin\varphi$ and $M = 6\sin\varphi(3\text{-}\sin\varphi)$. Oedometer tests also allowed c_v to be determined experimentally for a set of different stress levels in kPa and interpolated empirically in m^2/year as:

$$c_v = \sqrt{\left(1 + 0.14\sigma_v^{'}\right)} \tag{6}$$

The permeability k was assumed to be isotropic and was estimated as

$$k = \gamma_w c_v m_v = \gamma_w c_v \lambda / \sigma_v^{'} \tag{7}$$

where γ_w is a unit weight of water and m_v is the coefficient of volume compressibility of soil.

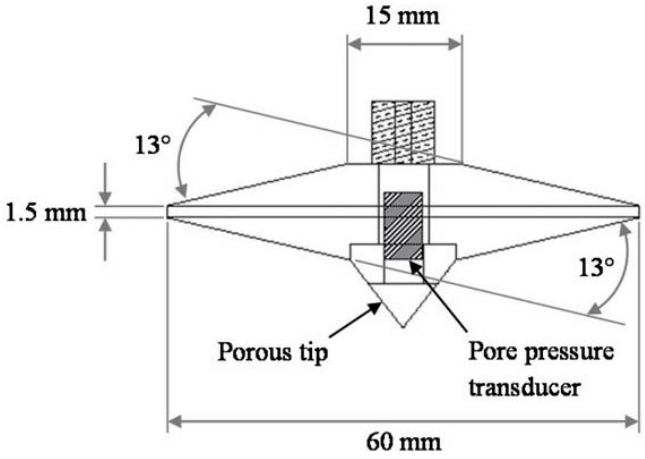
The geometry of the soil was modelled with approximately 3000 elements, expanding the domain limits to 20D to eliminate any chance of undesired boundary effects. A finer mesh was adopted around the spudcan. Due to the complexity of reproducing the penetration near the soil surface, an initial embedment of the spudcan of 0.5D was necessary. Penetration rate of the footing was 0.2 mm/s, similar to that in the experimental cases. The cases studied are summarized in Table 2.

EXPERIMENTAL SET-UP AND PROCEDURE

All the data presented were acquired in the drum centrifuge at the University of Western Australia. The drum centrifuge has a rotating channel that allocates a soil sample 160 mm deep. Centrally, a tool table that holds the actuator with the relative equipment is mounted and can be manoeuvred independently of the rotating channel, allowing for equipment maintenance without interrupting the rotation of the channel, i.e., disturbing the sample (Stewart et al. 1998[18] for more details). Centrifuge testing at an acceleration of 200 g facilitated the study as it allowed for several periods of consolidation to be investigated in a relatively short time. The consolidation period is normalised as $T = c_v t/D^2$, where c_v is the vertical coefficient of consolidation, t the dimensional period of consolidation and D the footing diameter at its widest section.

The spudcan model used in these tests is the same as that of Barboza-Cruz (2007)[4], and its geometry is schematically described in Figure 5. The widest section of 60 mm represents a diameter D = 12 m in the prototype.

Figure 5: Spudcan model schematic, Barbosa-Cruz (2007)[4].



The centrifuge channel was filled at 20 g with a kaolin clay slurry prepared with a water content twice the liquid limit and consolidated in flight for several days, keeping the water table above the soil surface and offering a two-way drainage path guaranteed by a geotextile at the bottom of the sample. T-bar penetration (section 5x20 mm) gave a shear strength profile $s_u = 1.1\ w$ kPa (w depth in m, prototype scale), whereas an average effective unit weight $\gamma' = 7.5$ kN/m^3 was determined from miniature piston core samples taken after the sample.

All the tests performed a penetration to a depth of 120 mm (2D) at a rate of 0.2 mm/s, which was fast enough to guarantee undrained conditions as $vD/c_v > 30$ (Finnie and Randolph, 1994[19]). In the tests involving consolidation, the actuator was switched from displacement to load control when the widest section of the spudcan reached $w = 60$ mm (1D), holding the load for four different t as reported in Table 2 before restarting the penetration in displacement control. A test with no pause in penetration called Exp-Ref was also performed as a reference case.

Table 2: Summary of numerical simulations and centrifuge tests reporting periods of consolidation at $w = 1D$.

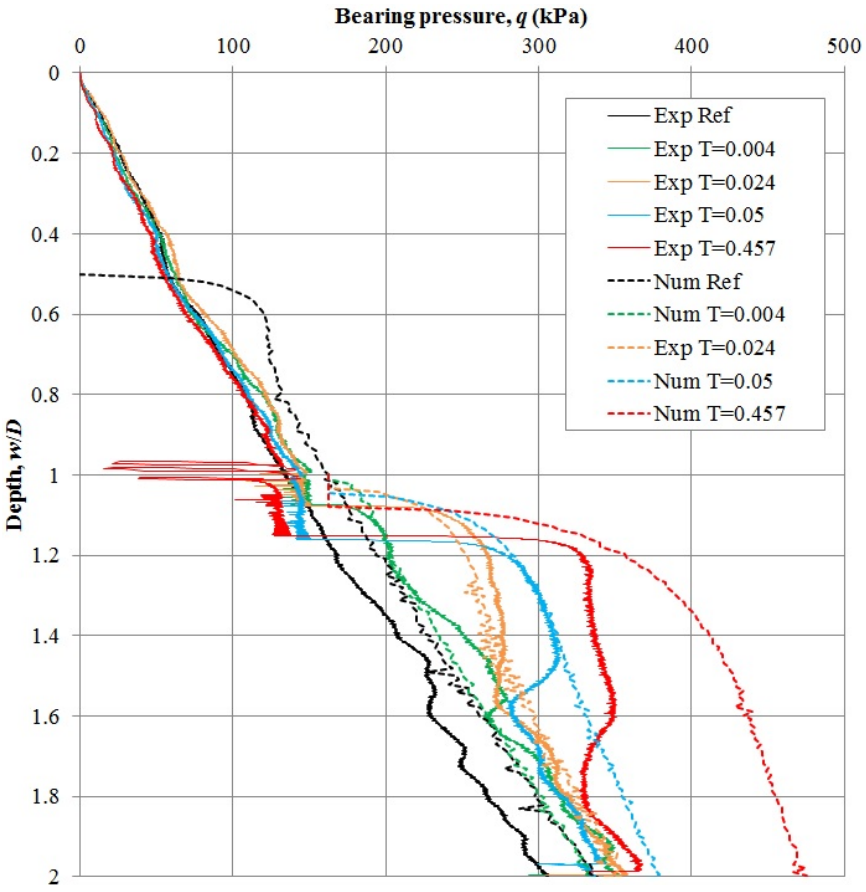
Num Simul	Exp Test	$T = c_v t/D^2$	t (prototype)
Num-Ref	Exp-Ref	n.a.	n.a.
Num -T15	Exp-T15	0.004	2 months
Num -T14	Exp-T14	0.024	1 year
Num -T13	Exp-T13	0.050	2 years
Num -T5	Exp-T5	0.457	18 years

RESULTS AND DISCUSSION

The results obtained from numerical simulations compared with centrifuge data are plotted in Figure 6, with dashed lines for the former and solid for the latter. The vertical profiles are presented in terms of the bearing capacity pressure $q = V/A$, where V is the force necessary to penetrate the spudcan and A the area at its widest section. Focusing on the behaviour post-consolidation, both cases show that a pause in the penetration, where the load is kept constant for a certain period of time, leads to an increase in q when the penetration restarts (Bienen

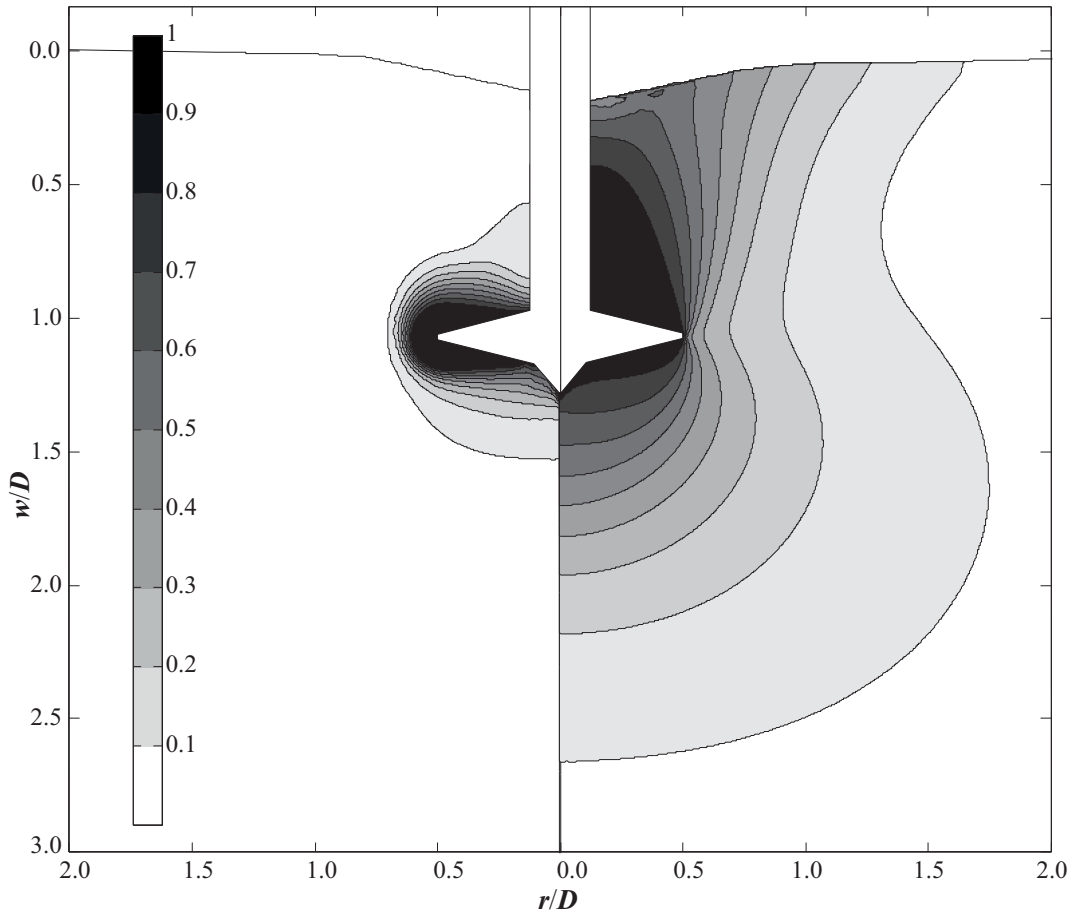
and Cassidy 2013[5], Stainer et al. 2014[20]). This behaviour can be explained if we consider the excess pore pressures that are built-up during the first stage of the installation due to the fast penetration rate of the spudcan. However, once the target depth is reached and consolidation is allowed to occur, the excess pore pressures gradually dissipate, with the velocity of the process regulated by the permeability of the soil. The dissipation causes a reduction in the void ratio of the soil around the spudcan and consequently an increase in the soil strength and stiffness.

Figure 6: Bearing capacity v w/D



To better understand what the failure mechanism is when this peak post-consolidation is observed, Figure 7 proposes numerical contour plots of the soil incremental displacements normalized by the spudcan displacement for the case immediately after consolidation, i.e., the first step of re-penetration in the FE simulation. The RHS shows the case with the longest $T = 0.457$, compared with the numerical reference case at the same depth on the LHS. With no stoppage in penetration, a fully localized mechanism is observed as the spudcan is already deep enough to guarantee such conditions (Hossain et al. 2005[2], Hossain and Randolph 2009[21]), with soil flowing around the footing. A completely different behaviour is observed in the case of consolidation, as previously experimentally presented in Stainer et al. (2014)[20] and Bienen et al. (2015)[22]. A much larger failure mechanism is shown, which extends as far as 1.75 D, 1.5 D below the spudcan and up to the soil surface. Suction is generated on the upper surface of the spudcan, causing a large cone of soil to be now involved in the mechanism. No longer is flow-around present, but instead the soil both above and below the spudcan moves downwards at a similar rate to the spudcan. This can be thought of as a newly enlarged foundation that is now being dragged and is generating a much higher resistance than the original size, for the consolidation causes this soil in the vicinity of the spudcan to strengthen and stiffen.

Figure 7: Contour plot of soil incremental displacement for reference case with no pause (LHS) and T = 0.457 immediately after consolidation (RHS).



Intuitively, the longer the period of consolidation, the more pronounced these effects will be. Figure 8 shows this increase in a similar fashion to that adopted in Bienen and Cassidy (2013)[5]. The ratio q_{peak}/q_{ref} is meant to quantify the magnitude of the phenomenon, q_{peak} being the pressure level reached after consolidation (approximatively 0.05 to 0.1 D after) and q_{ref} the same quantity at the same depth for the reference case. Although the trend is similar, a systematic underestimation of the numerical peak is observed compared to the centrifuge model data, with increment ratios ranging from 8% to 85% for the former and 25% to 98% for the latter. This may be because the strength recovery during consolidation cannot be captured by the hypoplastic model: Figure 9 shows an Abaqus view of the state variable sensitivity s at the end of the consolidation stage, immediately before re-penetration. Correctly, whereas all the soil far from the spudcan still preserves an intact structure (s = 2.2) as it is not subject to any particular deformation, in the vicinity of the spudcan and especially above it, the value dropped to $s \sim s_f = 1$ as a consequence of the spudcan shearing, which induced a large amount of remoulding (Hossain and Randolph 2009[21]). In reality, it is likely that the soil structure is partly recovered while the load is held constant, i.e., in centrifuge tests, there is a partial strength recovery, related to T, which the numerical model is still unable to capture.

Figure 8: Increase in bearing capacity q with non-dimensional consolidation time.

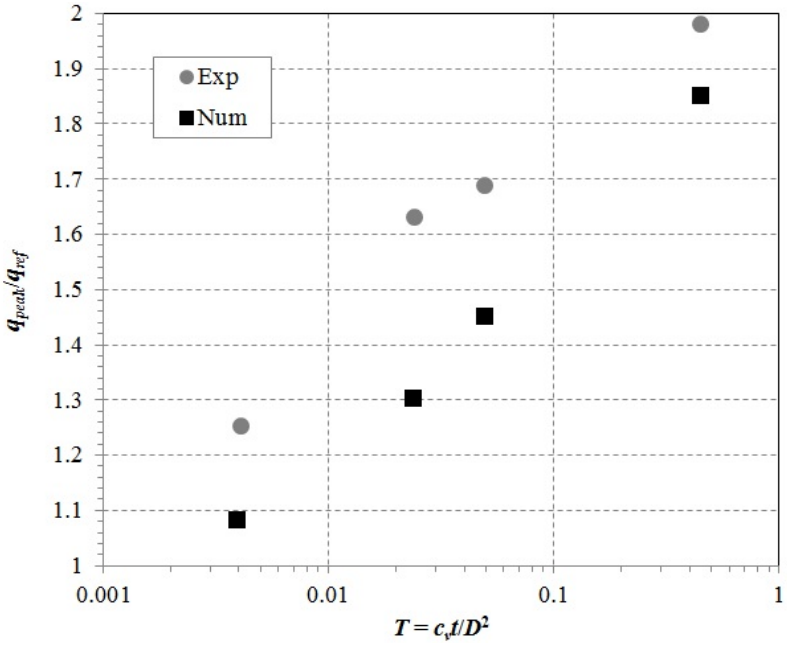
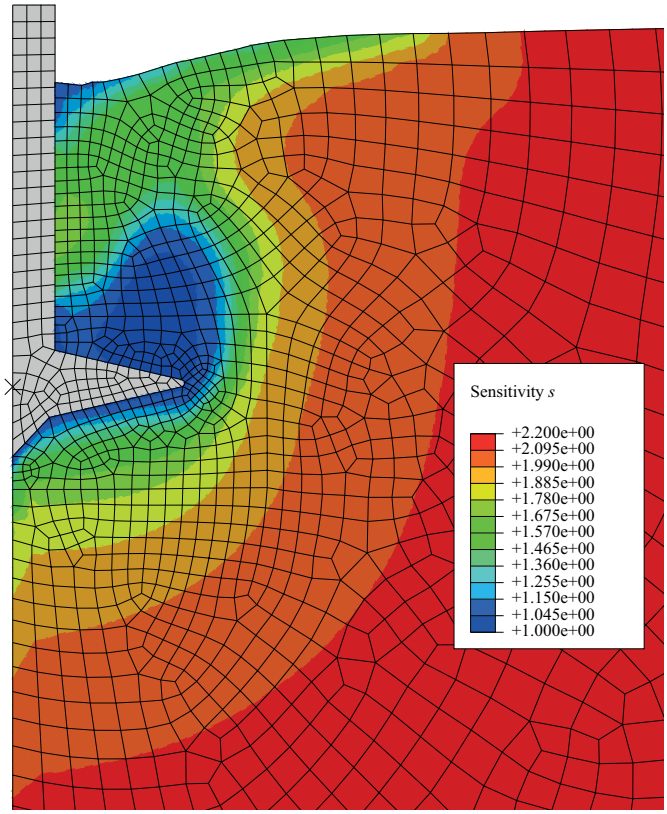


Figure 9: Abaqus/Standard contour plot of state variable sensitivity s after consolidation with T = 0.457.



OUTLOOK

The comparison shown in this paper offers a good agreement in terms of the behaviour post consolidation when looking at the two different approaches. However, there is still margin for improvement.

The gap in the penetration curve between the two reference cases (num-exp) is likely to be a consequence of the not fully completed process of consolidation for the centrifuge sample: this is in general left to consolidate in flight for several days, which however may not be sufficient to reach fully consolidated conditions. This is proven by the difference between $s_{u,exp}$ = 1.1 kPa/m and $s_{u,num}$ = 1.66 kPa/m: future centrifuge T-bar studies with several penetrations regularly spaced in time could offer an indication of when a final $s_{u,exp}$ value (closer to $s_{u,num}$) is approached.

With the introduction of the state variable s, soil softening can now be modelled. However, when consolidation is introduced into the problem, it brings along some strength recovery that the model does not have the capacity to model yet. Although the qualitative behaviour is similar, experimental and numerical re-penetrations do present a consistent difference in terms of q_{peak}/q_{ref}. Future efforts in coding a sensitivity s recovery could fill this gap and possibly also offer a better prediction of the entire post-consolidation penetration curve.

This study concentrated its efforts on a vertical one-dimensional penetration, showing an increment in the bearing capacity following a certain period of consolidation. These findings will be soon transferred into a three-dimensional VHM case, which includes vertical and horizontal displacements as well as rotations. Studies like those of Martin and Houlsby (2000)[23], Martin and Houlsby (2001)[24] and Bienen et al. (2006)[25] have proposed formulations of a rugby ball-shaped, three-dimensional yield surface, where the vertical load plays a vital role in the definition of its size. From an industry perspective, it would be certainly useful to understand whether a period of consolidation can be beneficial in the structure installation and in particular to help reduce the penetration depth, i.e., reaching the same target load at a shallower depth. For this reason, future research will study the effect of consolidation on the expansion of this yield surface as well as its potential reliability against multi-directional loadings representing large storm events that need to be assessed (SNAME (2008)[26], ISO (2012)[1]).

CONCLUSIONS

The effect of a pause in penetration during a spudcan installation was discussed. The paper presented the implementation of a hypoplastic constitutive model for structured clays in an LDFE analysis with the RITSS method, which allowed retrospective simulations of a centrifuge modelling test data on spudcan penetration-consolidation-penetration. The model was able to reproduce the peak in resistance following a certain period of consolidation caused by excess pore pressure dissipation and consequent soil strengthening, with the magnitude of this peak being proportional to the length of the pause. Contour plots of the soil incremental displacements demonstrated that the peak is also a consequence of a different failure mechanism: although a localized flow-around can be observed throughout the entire penetration with no stoppage, a much more extended one, including part of the soil being dragged downwards at the same rate at the spudcan, was observed after allowing a pause in penetration. Future studies will focus attention on the three-dimensional VHM case, trying to understand the effects of such consolidation on the expansion of the yield surface as well as its reliability against long term storm loading.

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