INTRODUCTION
Prior to the installation of a mobile offshore jack-up platform at a site, the load–penetration curve is required to be predicted. Typically bearing capacity theory is used at a series of embedment depths, with the soil response assumed to be drained in sand and undrained in clay (ISO, 2012). However, jack-up installation typically is a discontinuous process, with any pauses providing the opportunity for consolidation to occur in sufficiently permeable cohesive soils (such as silty clays and clayey silts), which results in a zone of increased strength of the soil and then an enhancement in the penetration resistance. Brennan et al. (2006) reported such ‘set-up’ behaviour even for relatively short durations of preloading holds of around 3–4 h. The penetration–consolidation–penetration response of spudcan footings has to date been investigated in experiments performed in centrifuges at the University of Western Australia (UWA, Barbosa-Cruz, 2007; Bienen & Cassidy, 2013; Stanier et al., 2014; Bienen et al., 2015). The existing centrifuge tests have limitations, as follows. (a) The load during the consolidation stage was maintained as the penetration resistance at the beginning of consolidation, which may not represent all practical scenarios. (b) The soil samples prepared were isotropic in permeability, while the permeability ratios of in-field naturally deposited sediments usually range between 1 and 5 (Vessia et al., 2012).

In this note, the entire ‘penetration–consolidation–penetration’ process of a spudcan in permeable clayey soil is investigated using a large-deformation finite-element (LDFE) method based on frequent mesh regeneration. The numerical model is validated by comparing with centrifuge tests of Purwana (2006) and Bienen & Cassidy (2013) for different kaolin clays. Further study is conducted to explore the effects of several key factors which are not covered in the previous tests. A simplified method for predicting the post-consolidation penetration resistance is proposed.

METHODOLOGY
The ‘penetration–consolidation–penetration’ of spudcan footings was investigated using an axisymmetric LDFE approach, remeshing and interpolation technique with small strains (RITSS), incorporating the modified Cam-clay (MCC) model. The details of the RITSS for coupled effective stress–pore pressure problems can be found in Wang et al. (2010, 2013, 2015). The soil close to the spudcan undergoes extreme and sudden changes in geometry in the very early stages of the installation process. To ensure computational convergence in the early stages of the LDFE analysis, the spudcan was pre-embedded at a shallow depth, \( D \approx 6D \), where \( D \) is the spudcan diameter. This was shown not to affect the results as sufficient penetration \( (0.3D \text{ minimum}) \) preceded the consolidation stage. During consolidation, the load imposed on the spudcan was held constant as a fraction \( \alpha \) of the resistance at the end of the first penetration stage. The shafted spudcan–soil interaction was simulated with frictionless contact (frictional contact yielded only slightly higher results). Drainage was permitted only at the soil surface. The soil horizontal and vertical dimensions were...
taken as no less than 20D to avoid boundary effects. The embedment depth of the spudcan was defined as the distance from the original soil surface to the lowest spudcan section with full diameter.

The UWA kaolin used in Bienen & Cassidy (2013) was normally consolidated, whereas the Malaysian kaolin in Purwana (2006) featured undrained strength between 5 and 10 kPa near the soil surface, owing to a surcharge applied during pre-consolidation of the sample at 1g, where g is the acceleration of gravity. In the reproduction of Purwana et al. (2005) tests the whole soil was simplified as normally consolidated. The MCC parameters of both soils are provided in Table 1. The undrained shear strength at depth z can be deduced from the MCC model (Wroth, 1984). The vertical permeability of the soil, kv, was expressed as function of the void ratio, as suggested in Mahmoodzadeh et al. (2014, 2015). Both kaolin samples are isotropic in permeability.

Significant research effort has been directed at demonstrating the effect of strain-softening and remoulding on the penetration resistance of penetrometers (e.g. Zhou & Randolph, 2009) and spudcans (Hossain & Randolph, 2009; Zhang et al., 2014). Hossain & Randolph (2009) quantify this influence at 10–15%. Zhang et al. (2014) illustrate separately the effect of large deformations (Fig. 1), comparing the wished-in-place results with those of ideal soil (soil sensitivity St = 1), and soil with strength degradation (St > 1). The former is accounted for in this study through the LDFE technique. The MCC model, however, does not consider strain softening. Accounting for this well-documented strength degradation through a reduction of 10% of the resistance obtained using the MCC model with intact su yields bearing capacity factors similar to those reported in Zhang et al. (2014) and, importantly, as obtained in centrifuge experiments (Figs 2 and 5 below). The undrained strength related to disturbance can be recovered, at least partially, during long-term consolidation, hence the strength degradation effect is not applied to the response immediately post-consolidation.

**COMPARISON WITH CENTRIFUGE TESTS**

*Tests by Purwana (2006)*

The centrifuge tests by Purwana (2006) were conducted at 100g. The spudcan diameter D was 125 mm in model scale. The coefficient of consolidation of Malaysian kaolin was obtained from oedometer test results (C. F. Leung, personal communication, 2014), fitted as

$$c_v = 5 \alpha \rho_o^m$$

where the units of cv and \(\alpha\) are m²/year and kPa, respectively.

The undrained strength profile measured was su = 1 442 kPa, close to 1 462 kPa from the MCC. St = 1.5. A test with consolidation duration of 1.16 years and consolidation load ratio \(\alpha = 0.75\) at depth of 1.5D was reproduced.

The numerical and experimental results are compared in Fig. 2. F is the net penetration resistance, which represents the total force applied on the spudcan, F, subtracting the submerged weight of the spudcan in soil. Beyond the over-consolidated crust (experiments) and the initial embedment (numerical), the experimental curve matches well with that from the coupled LDFE approach.

The excess pore pressures at different positions are shown in Fig. 3. At mid-radius of the spudcan base (position P3),
Tests by Bienen & Cassidy (2013)

The coefficient of consolidation of UWA kaolin was (Richardson, 2007)

\[ c_v = \sqrt{1 + 0.14\sigma} \tag{2} \]

An undrained strength gradient of 1.1 kPa/m was obtained from three T-bar tests before any spudcan tests. Soil sensitivity measured was 72. An effective unit weight of 6 kN/m³ was assumed, hence \( s_v = 146; \) kPa results from the MCC parameters. The spudcan was penetrated to a depth \( H = 1.5D \) and then the full load was held (\( \alpha = 1 \)) for consolidation. In Fig. 5, the capacity factor in this study achieves reasonable agreement with the experimental data. Load control in the centrifuge tests, especially during the early stages of consolidation, relies on feedback loops which may result in some load cycling and generation of additional excess pore pressures and settlement. It is therefore not surprising that the settlements predicted are smaller than those measured, as in Fig. 6.

The post-consolidation capacities and capacity factors predicted are shown in Fig. 7. The consolidation duration is normalised as

\[ T = t_c/D^2 \tag{3} \]

where \( t_c \) represents the value corresponding to the depth at the end of consolidation which is slightly larger than \( H \). The soil underneath the spudcan undergoes reduction of void ratio during consolidation, the undrained shear strength is thus improved (Fig. 8). As a consequence, the post-consolidation resistance is enhanced sharply within a minimal penetration and then the resistance profile moves towards the reference profile (without consolidation) gradually. This tendency can be observed more clearly in Fig. 7(b), in which the capacity factor is increased to a peak value and then reduced.

The effect of the consolidation duration on the post-consolidation resistance may be quantified through a ratio, termed peak ratio. \( N_{cp}/N_{cr} \) is the peak capacity factor during post-consolidation penetration, and \( N_{cr} \) represents the capacity factor at the same depth on the reference profile accounting for strain softening (see Fig. 7(b)). The reference profile accounting for softening is obtained by reducing the numerical reference resistance by 10%, as soil sensitivity is ~ 2. The predicted peak ratios are compared with three groups of centrifuge tests in Fig. 9. Both the experimental and numerical data highlight that the variation of the peak ratio with the normalised time depends insignificantly on the consolidation depth. The numerical predictions achieve reasonable agreement with the experimental results, although the trends with consolidation time differ moderately in their gradients. Neither set of data is perfect: on one hand, the centrifuge test results are affected by less than ideal load control, which is expected to have a stronger influence at shorter consolidation durations; the numerical analysis, on the other hand, essentially assumes remoulding of the soil is fully recovered during consolidation. Therefore, both may overestimate the peak ratio for short durations of consolidation (although for different reasons). Based on the LDFE analysis results and the above modelling considerations, a polyline is plotted in Fig. 9.

INFLUENCES ON PEAK PENETRATION RESISTANCE

The normalised consolidation duration and its influence on peak capacity factor is further explored numerically in terms of different soil permeabilities and consolidation load.
ratios. The horizontal permeability is taken as \( k_h = n k_v \) and the permeability ratio, \( n \), may be larger than 1.

The consolidation and further penetration in Malaysian kaolin is simulated, with \( n = 1 \) and \( \alpha = 1 \). Although \( c_v \) of Malaysian kaolin is \( \sim 10 \) times that of UWA kaolin, the peak ratios of both soils are close (Fig. 10), suggesting that the peak ratio in different soils can be approximated as a function of \( T_c \). The scenarios with \( H/D = 1 \) in UWA kaolin are repeated with \( n = 5 \). As the dissipation around the spudcan is influenced by both \( k_h \) and \( k_v \), equation (3) is updated to

\[
T_c = \frac{1 + n c_v \epsilon_c}{2 D^2}
\]

As shown in Fig. 10, the normalised consolidation duration defined in equation (4) quantifies the variation of the peak ratio for \( n = 1 \) and 5 well.

While Figs 7–10 are presented in terms of \( \alpha = 1 \), the peak ratios for \( \alpha = 0.5 \) and 0.75 are demonstrated in Fig. 11. The enhancement of the peak capacity factor increases with the consolidation load ratio at a given \( H/D \).

**PREDICTION OF POST-CONSOLIDATION RESISTANCE**

A simple procedure is proposed to predict the post-consolidation resistance, with the load ratio of unity as an example (the effect of a load ratio other than unity can be estimated using Fig. 11)

\( a \) Estimate the consolidation settlement corresponding to a given consolidation duration. The typical settlement can be derived from the numerical curves in Fig. 6.
CONCLUSIONS

A simple method is proposed to estimate the post-consolidation penetration resistance of spudcan footings that have experienced a pause during installation. The development is based on results from coupled pore fluid–effective stress large-deformation numerical analyses, with validation against centrifuge experimental data in normally consolidated kaolin clays. The numerical approach allowed the key factors influencing the consolidation-induced zone of increased undrained shear strengths to be investigated in detail.

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NOTATION

- $A$: projected area of spudcan
- $c_v$: coefficient of consolidation
- $D$: diameter of spudcan
- $e$: void ratio
- $F$: total penetration resistance
- $F'$: net penetration resistance
- $g$: acceleration of gravity
- $H$: spudcan depth at beginning of consolidation
- $H_{cp}$: post-consolidation penetration distance to mobilise $N_{cpv}^*$
- $K_r$: coefficient of earth pressure at rest
- $k_h, k_v$: horizontal and vertical permeability
- $N_c$: bearing capacity factor
- $N_{cpv}^*$: peak capacity factor post-consolidation
- $N_m$: reference capacity factor
- $n$: permeability ratio ($= k_h/k_v$)
- $S_i$: soil sensitivity
- $s_u$: undrained shear strength of soil
- $T$: normalised consolidation time
- $T_c$: normalised consolidation duration
- $t$: consolidation duration
- $z$: soil depth
- $\alpha$: consolidation load ratio
- $\alpha'$: vertical effective stress
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