A plasticity model for spudcan foundations in soft clay

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ABSTRACT

A plasticity model for predicting the load-displacement behaviour of a typical spudcan foundation for offshore jack-up platforms under combined vertical, horizontal and moment loading in soft clay is presented. Results from geotechnical centrifuge experiments of a spudcan embedded vertically to 0.7, 1.0 and 1.45 footing diameters are described. Augmented with finite element results these centrifuge experiments are used to evaluate the plasticity model components. As a result of soil backflow on top of the spudcan, enhanced combined bearing capacity was measured and this is reflected in increased yield surface size. A tensile vertical load capacity is also incorporated. The excellent predictive capabilities of the model are demonstrated by retrospectively simulating a selection of centrifuge tests.

INTRODUCTION

Mobile jack-up platforms are widely used for performing drilling activities in the offshore industry. A modern jack-up platform typically consists of three independent legs. Under each leg, a circular saucer shaped foundation known as spudcan (Figure 1) is embedded in the seabed, supporting the superstructure. Due to the environmental loading from wind, wave and current, spudcan foundations are subjected to complex combined vertical (V), horizontal (H) and moment (M) loading (see Figure 1 for the sign conventions for positive loads and conjugate displacements adopted in this paper). A jack-up is deployed multiple times throughout its life. Before each installation at a new site, a site specific assessment must demonstrate its stability in large storms (SNAME 2008; ISO 2012). This requires understanding of the bearing capacity of each spudcan as well as their interaction with the jack-up structure.

Figure 1. A typical spudcan foundation under combined *VHM* loading and corresponding displacements

Traditionally, spudcan footings have been simply treated as pin joints or a set of springs in the jack-up structural analysis, with foundation ‘failure’ under the storm evaluated separately using capacity surfaces written in terms of the *VHM* loads on the spudcan (see SNAME 2008, ISO 2012 for example). A significant improvement has been the development of force resultant foundation models that encapsulate the behaviour of the spudcan and surrounding soil within a plasticity framework (Schotman 1989; Nova and Montrasio 1991; Dean et al. 1997; Gottardi et al. 1999; Martin and Houlsby 2000, 2001; Cassidy et al. 2002a; Houlsby and Cassidy 2002; Bienen et al. 2006). These can be readily incorporated into a finite element structural analysis program as ‘macro elements’ as they are written directly in terms of force resultants and displacements about the spudcan’s load reference point (*LRP*, Figure 1). This avoids any complex discretization of the soil. Such integrated foundation-structure analyses of mobile jack-up platforms can be found, amongst others, in Thompson (1996), Williams et al. (1998), Martin and Houlsby (1999), Cassidy et al. (2001, 2002b), Houlsby and Cassidy (2002), Bienen and Cassidy (2006, 2009) and Vlahos et al. (2011).

The majority of force-resultant models have been based on single surface displacement-hardening plasticity theory. These have four components:

- (i) a yield surface written directly in *VHM* load space;

- (ii) a hardening law that establishes the yield surface size as a function of the plastic vertical footing displacement (w_p);
- (iii) an elastic matrix for describing any increment of load within the yield surface; and
- (iv) a flow rule to describe the elasto-plastic displacements during incremental expansion and contraction of the yield surface.

The current state-of-the-art model for predicting spudcan load-displacement behaviour in clay was developed by Martin and Houlsby (2000, 2001) and based on a series of 1 g laboratory tests in over-consolidated clay. No significant backflow was present in the tests even at relative large embedment depth of 1.6 footing diameters. Therefore, the model's yield surface only accounts for the contribution from the underside of the spudcan. However, in soft clays and at realistic field stress conditions, soil has been shown to encase the spudcan by flowing around the penetrating footing (Hossain et al. 2005; Hossain and Randolph 2009a) and this process has been codified in the new ISO industry guidelines (ISO 2012). The backflow soil, though remoulded during the installation process, potentially provides additional bearing capacity, especially for moment and horizontal loading. Furthermore, the backflow acts as a 'seal' on top of the spudcan, allowing suction to be developed, as shown in the centrifuge tests of Cassidy et al. (2004), Purwana et al. (2005) and Gaudin et al. (2011). This may considerably change the foundation behaviour under combined loading at very low or even tensile vertical load. Experimental push-over tests of a scaled model jack-up in clay have shown that a spudcan can sustain significant horizontal and moment reactions even under tensile vertical load (Vlahos et al. 2008).

To assess these potential benefits, the finite element method has been recently used to define the combined bearing capacity surfaces of 'wished in place' spudcan foundations with

complete backflow (Templeton et al. 2005; Templeton 2009; Zhang et al. 2011, 2012a) and enlarged surfaces primarily based on Templeton's work have been incorporated in the ISO guidelines (ISO 2012). Whilst the numerical studies confirmed an increase in combined loading capacity, there is a need to verify the magnitude of this increase experimentally. This is because soil undergoes significant displacement and remoulding during the spudcan installation. The effect of this was not considered in the numerical studies and the capacity is likely to have been overestimated.

The purpose of the research described in this paper was to i) experimentally measure the combined bearing capacity of a buried spudcan in soft clay and to ii) develop a force resultant foundation model that is appropriate for spudcan foundations in soft clay. Solutions for the relevant elastic behaviour have already been published (Zhang et al. 2012b) and these are used here. Further, a method to predict the vertical capacity in soft clay has been proposed by Hossain and Randolph (2009a; 2009b) based on combined numerical and experimental investigations and this forms the basis of the hardening law. However, a dedicated geotechnical centrifuge test programme was required to investigate the remaining components of (a) the yield surface in *VHM* space at various depths of embedment and (b) an appropriate flow rule. The centrifuge experiments reveal that the backflow considerably increases the footing's combined bearing capacity as compared to previous 1 g experiments results without backflow. The magnitude is, however, less than that predicted by the wished in place numerical analyses and, concerningly, as is recommended by the ISO guidelines.

APPARATUS

The tests were performed with a combined *VHM* loading apparatus developed recently for the geotechnical drum centrifuge at the University of Western Australia (UWA) (Zhang et al. 2013). The apparatus features independent displacement control of a model's in-plane vertical, horizontal and rotational degrees-of-freedom. Therefore, any combination of displacements can be prescribed to the model about its *LRP*. Figure 2 shows the assembly of the *VHM* loading apparatus with the model spudcan and loading arm attached. For the experiments described in this paper the model spudcan was 60 mm in diameter (D), corresponding to 12 m in prototype at the enhanced 200 g gravity level used in the tests. The loading arm consisted of a strain-gauged inner arm (used to measure the V , H and M forces acting solely on the spudcan and reported here at the *LRP*) and a stiff outer shield (that isolated the combined loads on the loading arm). A more detailed description of the spudcan, loading arm and test set-up are available in Zhang et al. (2013).

Figure 2. Assembly of the *VHM* load apparatus (after Zhang et al. 2013)

1 SOIL SAMPLE

2 The tests were performed in normally consolidated (NC) UWA kaolin clay (with properties
3 provided by Stewart 1992). To prepare the soil sample, the kaolin powder was mixed at
4 120% water content (twice the liquid limit) under vacuum and consolidated under self-weight
5 in the centrifuge at 200 g.

6
7 The soil was characterised by in flight miniature T-bar penetrometer tests (Stewart and
8 Randolph 1994). In each T-bar test, cycles of penetration and extraction over a depth of
9 20 mm (4 times the T-bar diameter) were performed until the penetration/extraction
10 resistance did not change with further cycles (i.e. fully remoulded). Such a cyclic event is
11 illustrated in Figure 3 for one test, whereas only the initial penetration is shown for the
12 remaining tests. The undrained shear strength (s_u) was derived using a typical T-bar factor of
13 $N_{T\text{-bar}} = 10.5$ (Einav and Randolph 2005; Dejong et al. 2011). The depth axis is shown in
14 prototype scale. A nonlinear expression of $s_u = 0.849w + 0.02w^2$ provides a good fit to the
15 undrained shear strength profile. The slight nonlinearity is attributable to linearly increasing
16 gravity field over the sample depth in the drum centrifuge. Previous studies (Stewart 1992;
17 Randolph and Hope 2004) have suggested an empirical normally consolidated strength ratio
18 $s_u/\sigma_{v0}' = 0.185$ for the UWA kaolin clay. In Figure 3, this empirical prediction is shown to
19 match the T-bar results well (note that the changing gravity field and variation of effective
20 unit weight with depth were taken into account when calculating σ_{v0}').

Figure 3. Undrained shear strength profile determined from T-bar tests

Figure 4 illustrates the soil strength degradation pattern during the cyclic T-bar events. The degradation factor is defined as the resistance (penetration/extraction) encountered by the T-bar at the middle depth of the cycles normalised by the initial penetration resistance (taken as cycle 0.5, Randolph 2004). Good repeatability is found and the limiting degradation factor is 0.45, indicating a sensitivity (S_t) of $1/0.45 = 2.2$. This is consistent with other cyclic T-bar tests performed on UWA Kaolin clay (e.g. Hodder et al. 2010; Mana et al. 2012).

Figure 4. Soil sensitivity determined from cyclic T-bar tests

EXPERIMENTAL PROGRAMME

The experimental programme concentrates on swipe tests (e.g. Tan 1990; Gottardi et al. 1999; Martin and Houlsby 2000) and radial displacement tests (e.g. Gottardi et al. 1999; Byrne and Houlsby 2001; Bienen et al. 2006) to evaluate the yield surface and flow rule, respectively, of the spudcan at three embedment depths, $0.7D$, $1.0D$ and $1.45D$. In addition, two pure vertical penetration-extraction tests were performed. Table 1 and Table 2 summarise the swipe and radial displacement testing programmes respectively.

Swipe tests

In a usual swipe test, the footing is first penetrated to a prescribed vertical depth. This is immediately followed by an excursion of translation (u) or/and rotation (θ) while the vertical displacement is held constant. Commencing from the peak compressive vertical load, a path on the VHM yield surface is tracked. By carrying out a systematic series of tests with

different $u/D\theta$ ratios, a comprehensive picture of the *VHM* yield surface can be established,

as illustrated in [Figure 5](#).

To investigate the yield surface on the tensile side of vertical load, some swipe tests were performed commencing with a tensile vertical load. After penetration to the target depth, tensile vertical load was mobilised through application of a small upward displacement. The swipe was then performed with the vertical penetration held constant. The mobilisation of the initial tensile vertical load is assumed to be elastic (i.e. no loss of plastic penetration w_p), so that a swipe test commencing from the peak tensile vertical load will track a path on the yield surface corresponding to the original w_p . A swipe test commencing from an intermediate tensile vertical load will first respond elastically and then track along that surface after yield.

This assumption is illustrated in [Figure 5](#).

Figure 5. Schematic illustration of swipe test load paths (modified after Gottardi et al. 1999)

Radial displacement tests

In the radial displacement tests the footing was subjected to a path of a fixed $u:w$ or $D\theta:w$ ratio (after being penetrated to an initial vertical embedment). Within the plasticity framework, this represents continuous elasto-plastic expansion of the yield surface. The tests, therefore, provide a basis for the development of the flow rule.

Two types of radial displacement tests were performed with slightly different procedures, with the assumed load paths schematically illustrated in Figure 6 (noting that in a radial displacement test the displacements are prescribed and the loads measured). The load paths represented by solid lines in the figure will be achieved by the application of radial displacements immediately after purely vertical penetration of the footing to a pre-selected depth. Plasticity theory implies that the load path starts from pure vertical load and expands the yield surface as the test proceeds. However, if the spudcan is unloaded elastically to around zero vertical load following the initial pure vertical penetration before radial displacements are applied, the load path will increase linearly to the yield surface (corresponding to the maximum previously experienced w_p) before expanding the yield surface from that point on. These load paths are indicated in the figure by the dashed lines.

Figure 6. Schematic illustration of radial displacement test load paths

Each swipe or radial displacement test represented a vertical penetration test before the swipe or radial displacements were applied. In all tests, the pure vertical penetration/extraction of

the spudcan was performed at a rate of $\dot{w} = 0.2$ mm/s. Assuming a coefficient of consolidation (c_v) of $2.6 \text{ m}^2/\text{year}$ for the effective stress level of interest (Stewart 1992), the non-dimensional velocity $\dot{w}D/c_v$ was 146.2, which was larger than the threshold value of 10~30 and thus ensured undrained conditions (Finnie 1993; House et al. 2001; Chung et al. 2006; Cassidy 2012). For swipe tests and radial displacement tests, the maximum horizontal and rotational rates were limited to 0.4 mm/s and $0.4^\circ/\text{s}$ respectively, as detailed in Table 1 and Table 2.

Effect of soil consolidation in the centrifuge

Since this paper is concerned with the effects of the backflow on the foundation behaviour, and it is well-known that the disturbed backflow will regain strength with time due to dissipation of excess pore pressure (see Purwana et al. 2005, Gan et al. 2012 for discussion relevant to jack-ups), it is therefore important to ensure that the properties of the backflow was properly modelled in the tests. In the centrifuge, the linear geometries are scaled down by N , while the rate of consolidation is scaled up by N^2 (Taylor, 1995), where N is the centrifugal acceleration as a ratio of g . In the field, a full preload of a jack-up normally takes about 24-36 h (Purwana et al. 2005) and soil maintains undrained during the installation process. In the centrifuge, undrained penetration process was also ensured with the dimensionless group $\dot{w}D/c_v$ held above the threshold between a partial drained and undrained penetration, as discussed above. However, the time in prototype used in penetration is longer. For the deepest penetration of $1.45D$ in the test programme, the total penetration time in prototype was 40 days. This give an opportunity of possible consolidation of the backflow material after passage of the spudcan. However, a quick estimation with the calculation theory shows minimal consolidation will occur. It should be further noted that the backflow

1 soil in the vicinity of the spudcan, which is more important on the footing behaviour, will
2 have even less time to consolidate.

3

4 As described above, in the swipes or radial displacements tests, lateral and (or) rotational
5 movement were immediately applied after the vertical event (penetration, and unloading for
6 certain tests), this was to ensure that the backflow material does not have the chance to
7 consolidate.

8

9 From the above discussions, the test procedures taken in this program ensures that the
10 foundation behaviour measured in the centrifuge tests reflects the behaviour of the spudcan
11 immediate after preloading in the field. Although the regaining of the soil strength with time
12 is possible, it was considered conservative to neglect the time effects in design.

13

1 Table 1. Summary of the swipe testing programme followed

Test depth	Type	Test Name	$u/D\theta$	Swipe movement		Velocities	
				u (mm)	θ (°)	$\dot{u}$ (mm/s)	$\dot{\theta}$ (°/s)
0.7D	Swipes from peak compressive load	PCS1	∞	12	0	0.4	0
		PCS2	0.95	9	9	0.4	0.4
		PCS3	-0.095	-0.9	9	-0.04	0.4
		PCS4	-0.57	-5.4	9	-0.24	0.4
		PCS5	-1.15	-9	7.5	-0.4	0.33
	Swipes from intermediate tensile load	ITS1	∞	12	0	0.4	0
		ITS2	-0.095	-0.9	9	-0.04	0.4
1.0D	Swipes from peak compressive load	PCS6	∞	12	0	0.4	0
		PCS7	0.95	9	9	0.4	0.4
		PCS8	-0.095	-0.9	9	-0.04	0.4
		PCS9	-0.57	-5.4	9	-0.24	0.4
1.45D	Swipes from peak compressive load	PCS10	∞	12	0	0.4	0
		PCS11	0.95	9	9	0.4	0.4
		PCS12	-0.095	-0.9	9	-0.04	0.4
		PCS13	-0.57	-5.4	9	-0.24	0.4
		PCS14	-1.15	-9	7.5	-0.4	0.33
	Swipes from intermediate tensile load	ITS3	∞	12	0	0.4	0
		ITS4	-0.095	-0.9	9	-0.04	0.4
	Swipes from peak tensile load	PTS1	∞	12	0	0.4	0
		PTS2	0.95	9	9	0.4	0.4
		PTS3	-0.095	-0.9	9	-0.04	0.4
		PTS4	-0.57	-5.4	9	-0.24	0.4

2
3

1 Table 2. Summary of the radial displacement testing programme followed

Test depth	Type	Test Name	Radial ratio	Radial movement		Velocities	
0.7D	$u:w$ plane		$\tan^{-1}(u/w)$ (°)	u (mm)	w (mm)	$\dot{u}$ (mm/s)	$\dot{w}$ (mm/s)
	Radial tests after elastic unload	R1	45	9	9	0.2	0.2
		R2	78.7	12	2.4	0.4	0.08
	$D\theta:w$ plane		$\tan^{-1}(D\theta/w)$ (°)	θ (°)	w (mm)	$\dot{\theta}$ (°/s)	$\dot{w}$ (mm/s)
	Radial tests after elastic unload	R3	46.3	9	9	0.2	0.2
		R4	79.2	9	1.8	0.4	0.08
1.45D	$u:w$ plane		$\tan^{-1}(u/w)$ (°)	u (mm)	w (mm)	$\dot{u}$ (mm/s)	$\dot{w}$ (mm/s)
	Radial tests immediately after vertical penetration	R5	22.5	3.73	9	0.083	0.2
		R6	45	9	9	0.2	0.2
		R7	76.7	21.1	5	0.4	0.095
	Radial tests after elastic unload	R8	45	9	9	0.2	0.2
		R9	78.7	12	2.4	0.4	0.08
	$D\theta:w$ plane		$\tan^{-1}(D\theta/w)$ (°)	θ (°)	w (mm)	$\dot{\theta}$ (°/s)	$\dot{w}$ (mm/s)
	Radial test immediately after vertical penetration	R10	22.5	3.56	9	0.079	0.2
		R11	81.8	10	1.52	0.4	0.061
	Radial tests after elastic unload	R12	46.3	9	9	0.2	0.2
		R13	79.2	9	1.8	0.4	0.08

2

3 FORCE RESULTANT MODEL

4 Hardening law

5 The model assumes that the yield surface expands with the plastic vertical displacement of
6 the spudcan (w_p). As the yield surface will be expressed as a function of the pure vertical
7 capacity, an accurate description of the vertical capacity of the spudcan at a given plastic
8 embedment depth is required.

9

For an embedded spudcan buried in backflow soil, the total vertical capacity for the footing to resist compressive external load is:

$$V_{\text{total_C}} = V_0 + F_b \quad (1)$$

where V_0 represents the compressive vertical bearing capacity provided by the soil and F_b represents the buoyancy force on the spudcan. V_0 and F_b are calculated as:

$$V_0 = N_c A s_{u0} \quad (2)$$

$$F_b = \gamma' Vol \quad (3)$$

where s_{u0} is the undrained shear strength at the current plastic embedment depth, N_c is the vertical bearing capacity factor, A is the bearing area of the footing, γ' is the effective unit weight of the soil and Vol is the volume of soil displaced by the footing.

The total capacity for the spudcan to resist external tensile load is

$$V_{\text{total_T}} = V_T - F_b \quad (4)$$

where V_T is tensile capacity provided by the soil cohesive strength. Its ratio to the corresponding compressive capacity (V_T/V_0) is denoted as χ , which will be shown useful when defining the yield surface.

In all tests, backflow was observed immediately after the spudcan *LRP* (Figure 1) penetrated past the soil surface. This is a result of very low s_u ($\sim$ zero) at the NC soil sample surface. Figure 7 shows (i) the penetration-extraction response of the two vertical tests and (ii) the initial penetration of all 11 swipes that were performed at a depth of around $1.45D$ ($\sim 87\text{mm}$). The resistance (V) is the measured vertical capacity corrected for the buoyancy and shaft effect (unequal areas below and above the spudcan due to existence of the shaft, similar to correction needed for a CPT test (Randolph et al. 2007)). Therefore, it represents the net

capacity provided by the cohesive strength of the soil (i.e. V_0 in penetration and V_T in extraction).

The pure vertical tests show that a peak tensile capacity of about 60% of the compressive capacity ($\chi \approx 0.6$) is mobilised at an extraction of $\sim 10\%$ of the spudcan diameter (i.e. 6 mm in model scale). Upon the vertical movement reversal, the load-displacement response is approximately linear, indicating elastic behaviour. However, once the net vertical load becomes tensile, the stiffness begins to degrade and reaches zero as the peak tensile load is mobilised. Clearly this mobilisation process is elasto-plastic. Based on the vertical tests, swipes (results of which will be presented later) commencing from the peak tensile vertical load were performed after an extraction of 6 mm; whilst those commencing from intermediate tensile vertical load (60-70% of the peak) tests were performed following 2.5 mm of upward vertical movement.

Figure 7. Vertical load response during penetration and extraction

Figure 8 shows the derived vertical bearing capacity factor (V/As_{u0}) against the normalised penetration depth (w/D). As can be seen, it decreases rapidly at shallow depth and approaches an average value of about 10 from approximately $0.5D$, indicating formation of a continuous flow-around soil mechanism. The large value at shallower depth is due to the high local soil heterogeneity.

Figure 8. Vertical bearing capacity factor of the spudcan

Hossain and Randolph (2009a; 2009b) proposed a mechanism based method for predicting the vertical bearing capacity of spudcans in clayey soils. The method takes into account the different failure mechanisms at various stages of penetration and can consider the effect of penetration rate and soil sensitivity. If the penetration rate effect is neglected, the vertical bearing capacity factor is determined by

$$N_{cs} \text{ or } N_{cd} = (\delta_{rem} + (1 - \delta_{rem})e^{-3\epsilon_b/\epsilon_{ps}}) N_{cs_ideal} \text{ or } N_{cd_ideal} \quad (5)$$

where N_{cs} is the shallow bearing capacity factor for a general shear failure mechanism before backflow, N_{cd} is the deep bearing capacity factor associated with a fully localised flow-around mechanism, N_{cs_ideal} and N_{cd_ideal} are corresponding shallow and deep bearing capacity factors respectively in an ideally non-strain softening soil (i.e. $S_t = 1$), δ_{rem} is the inverse of the soil sensitivity S_t , ζ_b represents the average cumulative shear strain as experienced by the soil particles as they traverse the flow mechanism around the spudcan. Hossain and Randolph (2009b) suggested ζ_b to be 2.4. ζ_{95} is the ductility parameter, which is equal to the cumulative strain required for 95% remoulding. Hossain and Randolph (2009a) recommend N_{cs_ideal} to be determined by interpolation of results from wished in place numerical analyses with an open cavity and N_{cd_ideal} to be calculated by

$$N_{cd_ideal} = 10 (1 + 0.065w_p/D) \leq 11.3 \quad (6)$$

Between the onset of backflow to full localisation of the flow-around mechanism, a transitional depth of $0.5D$ is assumed in this model. The transition reflects the physical response and avoids numerical difficulty through smoothing of the hardening law, which would otherwise exhibit a discontinuity. The bearing capacity in the transition zone is linearly interpolated. A small cavity depth of $0.05D$ is predicted by the Hossain and Randolph (2009a) method for the NC soil profile (for numerical reasons, the undrained shear strength at the surface is assigned a value of 0.1 kPa). Figure 8 shows the model's prediction of the bearing capacity factor based on $S_t = 2.2$ (as derived for the experimental conditions from cyclic T-bar tests) and an assumed ζ_{95} value of 30. A reasonable match to the experiments is demonstrated.

1 Yield surface

2 *Selected swipe test results*

3 **Figure 9** presents the load paths of three similar swipes of $u/D\theta = 0.95$ performed at
4 $0.7D$, $1.0D$ and $1.45D$ from peak compressive vertical load (PCS2, PCS7 and PCS11). With
5 increasing embedment depth, the load paths tracked by the swipes increase in magnitude.
6 However, a similar shape is maintained. This illustrates nicely that the size of the yield
7 surface expands as the current plastic penetration (w_p) increases and that a simple hardening
8 law can be assumed.

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Figure 9. Load paths of similar swipe tests (PCS2, PCS7 and PCS11) performed at different embedment depths

9
10
11
12 Figure 10 presents the load paths of three pure translation swipes (i.e. $u/D\theta = \infty$; PCS10,
13 PTS1 and ITS3) that were performed from peak compressive, peak tensile and intermediate
14 tensile vertical loads respectively at an initial embedment depth of $1.45D$ (note only the
15 limiting loads for test ITS3 are presented). The loads have been normalised by the

1 compressive capacity prior to the swipe (V_0). For the tensile swipes, V_0 refers to the
2 maximum compressive vertical load prior to the extraction. The implication of this
3 normalisation is that purely elastic behaviour during this initial pull out is assumed (or
4 alternatively the loss of w_p during the mobilisation of tensile load is neglected). This
5 assumption of the numerical model provides a simple and practical approach to evaluate and
6 eventually simulate the tensile swipe test results. As shown in Figure 10, the pair of swipes
7 performed from the peak compressive and peak tensile vertical loads track paths from two
8 ends of the VHM surface at a similar M/DH ratio. The limiting load combination of the
9 intermediate tensile swipe is also consistent with the yield surface established by the swipes
10 performed from the peak vertical loads.

Figure 10. Load path of similar swipes (PCS10, ITS3 and PTS1) performed from different initial vertical load levels at $1.45D$

Yield surface formulation

The following equation provides a reasonable and practical fit to the experimental data:

$$f = \left(\frac{H}{h_0 V_0} \right)^2 + \left(\frac{M/D}{m_0 V_0} \right)^2 - \frac{2eHM/D}{h_0 m_0 V_0^2} - \left(\frac{4}{(1+\chi)^2} \right)^2 \left(\frac{V}{V_0} + \chi \right)^2 \left(1 - \frac{V}{V_0} \right)^2 = 0 \quad (7)$$

where h_0 and m_0 define the peak ratios of H/V_0 and M/DV_0 in the VH ($M = 0$) and VM ($H = 0$) planes, e defines the eccentricity of the cross-section in the HM plane and χ is the tensile capacity ratio V_T/V_0 .

For the convenience of optimisation, a generalised non-vertical force is defined as:

$$Q = \sqrt{\left(\frac{H}{h_0} \right)^2 + \left(\frac{M/D}{m_0} \right)^2 - \frac{2eHM/D}{h_0 m_0}} \quad (8)$$

allowing the yield surface to be simplified to:

$$f = \left(\frac{Q}{V_0} \right)^2 - \left(\frac{4}{(1+\chi)^2} \right)^2 \left(\frac{V}{V_0} + \chi \right)^2 \left(1 - \frac{V}{V_0} \right)^2 = 0 \quad (9)$$

By using Eq. 8, the load path of a swipe test in the VHM space is simplified to a path in the VQ plane. Least squares regression analysis was performed for each complete set of compressive swipe tests at different depths. As the pure vertical tests indicate a tensile capacity ratio of 0.6, and indeed the best-fit χ values found by optimisation are around 0.6 (Zhang et al. 2013), the χ value is held at 0.6 for simplicity. Table 3 collates the optimised parameters for the three embedment depths and Figure 11 compares all the 22 swipe tests against Eq. 9 in the VQ plane (note only the limiting loads are shown for the four intermediate tensile swipe tests). It is illustrated that all the compressive swipes bond closely around the fitted surface.

1 Although the tensile swipes are presented in Figure 11, they were not included in the fitting
2 process. Nonetheless, tests PTS1 and PTS2 with $u/D\theta \geq 0$ compare closely to the surface
3 determined by the compressive swipes. However, tests PTS3 and PTS4 with $u/D\theta < 0$ locate
4 inside the yield surface. Similar observations can be made for the swipes performed from
5 intermediate tensile load (i.e. tests ITS1 and ITS3 with $u/D\theta \geq 0$ reach limiting loads that are
6 consistent with the fitted surface, but, tests ITS2 and ITS4 with $u/D\theta < 0$ fall inside of the
7 surface). No conclusive explanation for this can be offered at this stage.

8 **Table 3. Summary of best-fit parameters.**

Parameters	h_0	m_0	e	
$0.7D$	0.182	0.089	0.380	
$1.0D$	0.198	0.092	0.244	0.6
$1.45D$	0.242	0.094	0.150	

9

10

11 **Figure 11. Swipe tests against the fitted surface in the VQ plane**

12

13 To further illustrate the fit, the load paths of swipe tests performed from peak compressive
14 and peak tensile vertical loads at $1.45D$ are projected onto the HM plane in Figure 12a and

Figure 12b respectively. Data points at discrete vertical load levels ($V/V_0 = 0.8, 0.7, 0.6, 0.5$ and 0.4) for the compressive swipes and ($V/V_0 = -0.5, -0.4, -0.3$ and -0.2) for the tensile swipes are compared with corresponding cross-sections of the fitted yield surface. As a load combination (V, H, M) has the same effect as $(V, -H, -M)$ on the footing, the experimental data is also plotted in the diagonally opposite quadrant. The fitted surface describes the compressive swipes very well. The yield surface also compares closely to the data of the tensile swipes in the $(+H, +M)$ and $(-H, -M)$ quadrants. However, it is un-conservative compared to the data in the $(-H, M)$ and $(H, -M)$ quadrants. A better fit to the tensile swipes is possible by performing a separate regression analysis on those test data, but will require a different set of yield surface parameters. In practical situations, the moment on the foundation is often caused by the environmental load (H) acted at an elevated height. Therefore, H and M are normally of the same sign, i.e. locate in the $(+H, +M)$ and $(-H, -M)$ quadrants. For simplicity, the yield surface adopts the best fit parameters evaluated from the compressive swipes.

(a) Swipes from peak compressive vertical load

(b) Swipes from peak tensile vertical load

Figure 12. Experimental data against the fitted yield surface in the normalised HM plane

Discussion on the yield surface

Figure 13 compares the cross-sections of the yield surface in the normalised VH ($M = 0$) and VM ($H = 0$) planes derived from (i) the current centrifuge tests, (ii) the 1 g experiments of Martin and Houlsby (2000), and (iii) the finite element analyses of Zhang et al. (2011) of a wished in place spudcan with complete backflow. On these two planes, Eq. 7 indicates a parabolic interaction envelope, similar to the 1 g surface. However, the location of the peak differs. The envelope based on the centrifuge results peaks at $V/V_0 = 0.2$, implying significant combined bearing capacity even under tensile vertical load. The envelope developed from 1 g testing peaks at an intermediate vertical load level ($V/V_0 \approx 0.46$) and the capacity diminishes to zero when the vertical load approaches zero. This is related to the different backflow and levels of suction between the centrifuge and 1 g tests, as also discussed in Vlahos et al.

(2008). The numerical analyses suggest elliptical interaction envelopes in these two planes (that differ in shape between the VH ($M = 0$) and the VM ($H = 0$) planes, and both are larger than the envelope evaluated from the centrifuge results). The numerically derived envelopes peak at zero vertical load due to the assumption of the bonded footing-soil interface and the Tresca yield criterion for the soil (thus the same tensile capacity as the compressive capacity would be calculated).

Figure 13. Comparison of yield surface shape in the normalised VH ($M=0$) and VM ($H=0$) planes

On the VH and VM planes, the current ISO guidelines recommend that the interaction envelope evolves progressively from a parabola similar to Martin and Houlsby's envelope (except that it peaks at $V/V_0 = 0.5$) for a surface spudcan to an ellipse for a spudcan with embedment ratio (w/D) equal to or greater than 2.5. However, the current centrifuge results seem not to support this evolution.

While the shape of yield surface is determined by the equation form (e.g. parabolic vs elliptical), the size of surface in the normalised load space is determined by the parameters h_0 and m_0 . In Figure 14, the centrifuge results show that the values of h_0 and m_0 increase with embedment, especially h_0 . This is in contrast to the 1 g test results, where h_0 and m_0 were

1 constant with depth. The centrifuge h_0 and m_0 values are also considerably higher, indicating
2 increased bearing capacity due to backflow. Although the wished in place numerical analyses
3 successfully capture this trend, the magnitude is overestimated as the remoulding and soil
4 displacement during the installation process are not accounted for. The ISO guidelines
5 recommend h_0 and m_0 values increase with embedment depth for spudcan in soft clay site
6 based primarily on limited wished in place numerical analyses (of Templeton et al. 2005;
7 Templeton 2009). Although a method is provided to correct for soil remoulding, the resulting
8 h_0 and m_0 values according to ISO are still overly optimistic when compared to these
9 experimentally measured values. This is shown in Figure 14, noting that the ISO data points
10 are based on $S_t = 2.2$ with assumption of fully remoulded backflow and a projected horizontal
11 area to vertical area ratio of 0.31, relevant to the centrifuge experimental conditions.

Figure 14. Variation of h_0 and m_0 with embedment depth

15 Figure 15 compares the eccentricity of the HM cross-sections of the yield surfaces derived
16 from the centrifuge tests, the 1 g experiments and the finite element analyses. The centrifuge

1 results show that eccentricity reduces with embedment, consistent with the findings from
 2 finite element analyses. Contrary to this, the eccentricity of the surface derived from the 1 g
 3 experimental does not change with depth. The footing essentially behaves like a surface
 4 foundation since no significant backflow was present in the tests.

Figure 15. Variation of eccentricity with embedment depth

8 Elasticity

9 Linear elastic response is assumed for load combinations inside the yield surface. The
 10 relationship between the increments of load (δV , δH , δM) and the corresponding elastic
 11 displacements (δw_e , δu_e , $\delta \theta_e$) is:

$$12 \begin{Bmatrix} \delta V \\ \delta H \\ \delta M \end{Bmatrix} = \frac{GD}{8} \begin{bmatrix} 4K_V & 0 & 0 \\ 0 & 4K_H & 2K_C D \\ 0 & 2K_C D & K_M D^2 \end{bmatrix} \begin{Bmatrix} \delta w_e \\ \delta u_e \\ \delta \theta_e \end{Bmatrix} \quad (10)$$

13 where G is the shear modulus of the soil at current plastic embedment (w_p) and K_V , K_H , K_M
 14 and K_C are dimensionless stiffness coefficients that have been already defined by the finite
 15 element analyses of Zhang et al. (2012b). The coefficients reported in Zhang et al. (2012b)

are representative of a spudcan shaped footing buried in backflow soil and provide for a range of linearly changing shear modulus profiles. The methodology followed several numerical studies defining the elastic stiffness of a footing under combined *VHM* loading (Bell 1991; Ngo-Tran 1996). Table 4 lists the representative stiffness coefficients relevant for the spudcan for the experimental soil conditions at different depths.

Table 4. Representative elastic stiffness coefficients (after Zhang et al. 2012b).

w/D	K_V	K_H	K_M	K_C
0.7	21.04	12.99	11.60	1.24
1.0	18.63	12.66	11.51	1.27
1.45	17.63	12.61	11.56	1.23

For interpretation of the radial tests results and in the retrospective simulations reported in this paper, the rigidity index $I_r = G/s_{u0}$ was evaluated based on the unload-reload stiffness observed in the vertical tests (and by using the dimensionless elastic stiffness coefficient reported in Zhang et al. (2012b) according to Eq. 10). An average value of $I_r = 50$ was found to fit the experimental data well.

Flow rule

The flow rule dictates the direction of plastic flow during an elasto-plastic increment. In this model, a plastic potential surface with a similar form to the yield surface is proposed:

$$g = \left(\frac{H}{\alpha_h h_0 V_0'} \right)^2 + \left(\frac{M/D}{\alpha_m m_0 V_0'} \right)^2 - \frac{2eHM/D}{\alpha_h \alpha_m h_0 m_0 V_0'^2} - \alpha_v^2 \beta_{12}^2 \left(\frac{V}{V_0'} + \chi \right)^{2\beta_1} \left(1 - \frac{V}{V_0'} \right)^{2\beta_2} = 0 \quad (11)$$

where

$$\beta_{12} = \frac{(\beta_1 + \beta_2)^{(\beta_1 + \beta_2)}}{\beta_1^{\beta_1} \beta_2^{\beta_2} (1 + \chi)^{(\beta_1 + \beta_2)}} \quad (12)$$

V_0' is the dummy V_0 value of the plastic potential surface that intersects the yield surface at the current load point (V, H, M) . The non-association parameters α_v , α_h , α_m , β_1 and β_2 are used

to adjust the shape of the surface. For $a_v = a_h = a_m = \beta_1 = \beta_2 = 1$, Eq. 12 is equivalent to Eq. 7, simulating associated flow.

In the following sub-sections, the development of the proposed plastic potential surface is explained.

Plastic flow in the HM plane

According to plasticity theory the ratio of plastic horizontal displacement to rotation can be evaluated as

$$\frac{\delta u_p}{D \delta \theta_p} = \frac{\partial g / \partial H}{\partial g / \partial (M/D)} = \frac{\left(\frac{\alpha_m}{\alpha_h} \right) \left(\frac{m_0}{h_0} \right) \left(\frac{H}{M/D} \right) - e}{\left(\frac{\alpha_h}{\alpha_m} \right) \left(\frac{h_0}{m_0} \right) - e \left(\frac{H}{M/D} \right)} \quad (13)$$

where δu_p and $\delta \theta_p$ are plastic horizontal and rotational displacement increments respectively.

When $a_h = a_m = 1$, Eq. 13 represents associated flow in the *HM* plane. In Figure 16, the measured plastic displacement ratios $\delta u_p / D \delta \theta_p$ in the swipe tests are compared with the prediction of associated flow (for clarity this is only shown for depth of $0.7D$; however, the results are similar for other depths). The experimental plastic displacements were obtained by subtracting the calculated elastic components from the prescribed total displacements. With the test data falling on both sides of the associated prediction, an associated flow appears a good first estimation, though the associated assumption consistently under-estimates the absolute plastic displacement ratio ($|\delta u_p / D \delta \theta_p|$) at a given DH/M ratio. However, when $a_h = 1.0$, $a_m = 1.35$, the predicted plastic displacement ratios match the measured data very well for all depths, as illustrated by the example shown in Figure 16 for $0.7D$.

Figure 16. Plastic flow in the *HM* plane

Plastic flow in the *VH* and *VM* planes

In the *VH* and *VM* planes, radial displacement tests were performed to evaluate the flow rule. Figure 17 presents the load paths of four radial tests (R8, R9, R12 and R13 in Table 2) at $1.45D$, which were performed from a net vertical force of around zero (achieved through 0.8 mm of upwards movement following initial penetration). The horizontal and moment loads on the spudcan are expressed as the generalised non-vertical force Q according to Eq. 8. Yield surfaces with different V_0 values are shown against the load paths, illustrating their expansion during the radial displacement tests.

Figure 17. Load paths and example yield surfaces of radial displacement tests

If associated flow is assumed,

$$\frac{\delta q_p}{\delta w_p} = \frac{\partial f / \partial Q}{\partial f / \partial V} \quad (14)$$

where δq_p is the generalised plastic displacement conjugate to Q , which can be expressed as:

$$\delta q_p = \sqrt{\frac{1}{1-e^2} \left[(h_0 \delta u_p)^2 + (m_0 D \delta \theta_p)^2 + 2e(h_0 \delta u_p)(m_0 D \delta \theta_p) \right]} \quad (15)$$

as demonstrated in Cassidy et al. (2002a).

In Figure 18, the plastic displacements of the radial displacement tests performed at both depths of $0.7D$ and $1.45D$ are compared with prediction of Eq. 14, which clearly shows that the associated flow does not describe the experimental data satisfactorily (Note R5, R6, R7, R10 and R11 are not shown for $1.45D$ for clarity, though the comparison is similar).

Based on the plastic potential surface of Eq. 13, least squares regression was performed to find the optimal set of parameters to describe the radial displacement test data. For

consistency with the HM plane, a_h and a_m are constrained to be 1.0 and 1.35 respectively. Further, β_1 and β_2 were constrained to be equal to ensure that the peak of the plastic potential occurred at $V/V_0' = 0.2$. Values of $a_v = 1.1$, $\beta_1 = \beta_2 = 0.45$ were found to predict the experimental data well, as illustrated in Figure 18.

Figure 18. Plastic flow indicated by radial displacement tests

RETROSPECTIVE ANALYSES

The force resultant model has been implemented into a FORTRAN program and retrospective simulations of selected centrifuge tests have been performed to demonstrate the capabilities of the model. As the entire centrifuge program was displacement controlled, the input to the numerical analysis consisted of displacements identical to those imposed in the experiments, with the FORTRAN program calculating the spudcan reactions according to the model.

Retrospective simulation of a pure vertical test

Figure 19 compares the numerical simulation of a vertical test with the centrifuge experiment. A reasonably good match is demonstrated. The assumption of purely elastic behaviour within the yield surface can be observed to be slightly simplistic during the start of the extraction

1 phase. A close simulation until $V = 0$ can be observed (as expected as the shear modulus G
 2 was fit from the experiments) but after that the experiments see a more non-linear reduction
 3 of stiffness than the model predicts.

4
 5 **Figure 19. Simulation of a pure vertical penetration-extraction test**
 6

7 *Simulation of swipe tests*

8 Due to space limitations only two swipe tests are shown. Figure 20 shows the simulation of
 9 swipe test PCS2 ($u/D\theta = 0.95$, $w/D = 0.7$) against the experimental results. Both the load
 10 paths and load-displacement response are predicted well by the numerical simulation.

Figure 20. Simulation of swipe test PCS2

Figure 21 shows the simulation of the swipe test PTS2 ($u/D\theta = 0.95$, $w/D = 1.45$), which was performed from peak tensile vertical load. It is demonstrated that the load path predicted by the numerical simulation matches the experimental response well. However, the numerical simulation terminates at a higher vertical load level, which is determined by the peak of the plastic potential or the ‘parallel point’ (Tan 1990; Cassidy et al. 2002a) where the load state does not change despite continuing deviatoric movements. According to the flow rule adopted in this model, the parallel point occurs at $V/V_0' = 0.2$ (the V/V_0 ratio at the termination point is also close to 0.2, with its exact value dependent on the load path).

Figure 21. Simulation of swipe test PTS2

Simulation of radial displacement test

Figure 22 shows the simulation of radial test R6, in which the spudcan was displaced at a fixed ratio of $u/w = 1$ after the vertical load was unloaded elastically to approximately zero. The model predicts that the loads on the spudcan will increase linearly (purely elastic behaviour) until reaching the yield surface established during the initial vertical penetration. The loads on the spudcan then move quickly along the expanding yield surface and finally “find” the location where the theoretical plastic displacement ratio matches the prescribed (after the small fraction of elastic components is deducted). However, the experimental load path does not have an obvious elastic-plastic transition point and this is a limitation in the single surface displacement hardening approach. A similar response is also reported for a caisson foundation in clay (Cassidy et al. 2006). After reaching the yield surface, the simulation expands the yield surface at a similar location of the yield surface as the experiment results. This demonstrates that the plastic flow rule correctly predicts the displacements ratios. However, the magnitude of expansion predicted by the model is less than in the experiment. In other words, the yield surface of the foundation under the displacement path of the radial test is greater than the prediction by the vertical displacement hardening law. Such phenomenon has also been observed in similar tests on sand (Byrne and Houlsby 2001; Bienen et al. 2006) and improvements to the model framework were made by including the plastic horizontal and rotational displacements into the hardening law (radial hardening). This achieved better simulations of the experiments. However, this is not pursued in the current model so as not to increase its complexity. It should be noted that the current model is conservative in terms of predicting the yield surface expansion by not incorporating the radial hardening.

Figure 22. Simulation of radial displacement test R6

CONCLUSIONS

This paper presents a comprehensive set of centrifuge swipe tests that measured the combined bearing capacity of a typical spudcan foundation in soft clay at three different embedment depths ($0.7D$, $1.0D$ and $1.45D$) under compressive as well as tensile vertical load. These tests confirmed that soil backflow (i) increases the size of the bearing capacity surface and (ii) provides significant bearing capacity under tensile vertical load. The size of the surface also increases with embedment depth. However, the magnitude of bearing capacity enhancement due to the backflow is less than the recommendations by the current ISO guidelines.

Although the ISO guidelines allow for advanced force resultant modelling in a jack-up assessment, no guidance on how to achieve this is provided. Therefore, the test results reported in this paper are used to develop a force resultant foundation model that describes the load-displacement behaviour of the foundation under combined VHM loading. The model uses existing elastic solutions based on finite element analyses and a hardening law derived

from combined experimental and numerical investigations. The bearing capacity surface evaluated from the swipe tests forms the yield surface of the plasticity model. Those swipe tests also reveal slightly non-associated plastic flow in the *HM* plane. Additional centrifuge radial displacement tests were performed to investigate the plastic flow rule in the *VH* and *VM* planes and demonstrated significant non-associated behaviour in those two planes. An appropriate flow rule that scales the shape of the yield surface has been proposed for the model.

The model predicts the general foundation behaviour well and is appropriate for predicting the pushover capacity of a mobile jack-up during a site-specific assessment analysis. Options for model improvement have been highlighted, though these will increase model complexity. The model has been developed from monotonic loading experiments and further developments are required for the model to capture the response to cyclic loading.

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1 LIST OF NOTATIONS

2	A	vertical bearing area of the spudcan
3	c_v	coefficient of consolidation
4	D	diameter of the footing
5	e	eccentricity of the HM cross-sections of the yield surface
6	F_b	buoyancy force
7	G	soil shear modulus
8	H	horizontal load on the footing
9	h_0	peak ratio of H/V_0 in the VH ($M = 0$) plane
10	I_r	rigidity index
11	K_V, K_H, K_M, K_C	dimensionless elastic stiffness coefficients
12	LRP	load reference point of the footing
13	M	moment load on the footing
14	m_0	peak ratio of M/DV_0 in the VM ($H = 0$) plane
15	N_c	vertical bearing capacity factor
16	N_{cd}	deep vertical bearing capacity factor
17	N_{cd_ideal}	deep vertical bearing capacity factor in an ideally non-strain softening soil
18	N_{cs}	shallow vertical bearing capacity factor
19	N_{cs_ideal}	shallow vertical bearing capacity factor in an ideally non-strain softening soil
20	N_{T-bar}	bearing capacity factor for the miniature T-bar penetrometer
21	Q	generalised non-vertical force
22	q_p	plastic non-vertical displacement conjugate to Q
23	S_t	sensitivity of the soil
24	s_u	undrained shear strength of the soil
25	s_{uo}	undrained shear strength at the current plastic embedment depth of the footing
26	u	horizontal footing displacement
27	u_e	elastic horizontal footing displacement
28	u_p	plastic horizontal footing displacement
29	$\dot{u}$	horizontal displacement velocity
30	V	vertical load on the footing
31	Vol	volume of soil displaced by the spudcan
32	V_0	peak vertical compressive capacity
33	V_0'	dummy peak vertical compressive capacity for the plastic potential surface
34	V_T	peak vertical tensile capacity

1	$V_{\text{total_C}}$	vertical capacity of the footing to resist compressive external load
2	$V_{\text{total_T}}$	vertical capacity of the footing to resist tensile external load
3	w	vertical footing displacement
4	w_e	elastic vertical footing displacement
5	w_p	plastic vertical footing displacement
6	$\dot{w}$	vertical penetration velocity
7	θ	rotation of the footing
8	θ_e	elastic rotation of the footing
9	θ_p	plastic rotation of the footing
10	$\dot{\theta}$	rotational velocity
11	χ	ratio of peak tensile capacity to peak compressive capacity
12	γ'	effective unit weight of the soil
13	σ_{v0}'	effective vertical consolidation pressure
14	δ_{rem}	the inverse of soil sensitivity
15	ξ_{95}	cumulative shear strain required for 95% remoulding
16	ξ_b	average cumulative shear strain experienced by the soil as it traverses the flow
17		mechanism around the spudcan
18	$a_h, a_m, a_v, \beta_1, \beta_2$	non-association parameters

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