Jack-up push-over analyses featuring a new force resultant model for spudcans
in soft clay

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Abstract

With the development of the offshore oil and gas industry, mobile jack-up drilling platforms are increasingly utilised in deeper waters and harsher environments. Excessive conservatism in jack-up site specific assessment may result in undue rejection of a unit. In soft clayey seabeds, the spudcan foundations of the jack-up platform penetrate deeply into the soil with partial or complete backflow. Although the deep penetration and associated backflow are widely perceived to increase horizontal and moment foundation bearing capacities in particular, the problem is not well understood. In this paper therefore, a plasticity foundation model is proposed that accounts for the effects of backflow. It has been developed through combined numerical and experimental study. The model is suitable for performing integrated soil-structure analyses. Results of such analyses of a jack-up under quasi-static push-over load are discussed to highlight the impact of the model in the context of site specific assessment of jack-up rigs in soft clay.

1 Introduction

In the offshore industry, mobile jack-up platforms (Figure 1a) are widely used for performing drilling activities. A modern jack-up platform is typically supported by three independent legs. Each leg is equipped with a circular saucer shaped foundation known as spudcan (Figure 1b). A jack-up is installed by preloading the platform to typically one and a half to twice the platform's self weight through ballasting the hull with sea water. The spudcan foundations penetrate into the soil under the preload. Once installation is completed, the ballast is dumped and the jack-up operates under its self weight and working variable loads. In addition, a jack-up platform is subjected to significant environmental loading from wave, wind and current. This transfers combinations of vertical (V), horizontal (H) and moment (M) loading to the spudcan footings (see Figure 1b for the sign conventions adopted for positive

loads and conjugate displacements). A jack-up is deployed multiple times throughout its life and before every installation at a new location a site specific assessment must demonstrate its stability in large storms (SNAME 2008; ISO 2012). This requires understanding of the bearing capacity of spudcans under combined *VHM* loading, as well as their interaction with the jack-up structure, which is influenced by the boundary conditions provided by the seabed soil.

Figure 1. A typical jack-up platform (after Reardon 1986) and a spudcan under *VHM* loading

In Figure 2, two idealized scenarios after spudcan installation in clay are schematically illustrated. Figure 2a shows a situation that occurs in stiff clay where an open cavity is formed above the spudcan. Failure is governed by a general shear mechanism. Figure 2b, on the other hand, illustrates complete soil backflow due to a continuous flow-around mechanism. The undrained shear strength of offshore soft clayey soil profiles often has a small non-zero value at the surface (less than 15 kPa) and increases linearly with depth at a

strength gradient (k) of typically in the range of 0.6 to 3.0 kPa/m (Tani and Craig 1995). In these soils, deep penetration of the spudcan footings is required to provide sufficient resistance, and soil backflow occurs after a critical depth of penetration (though backflow may not be complete, depending on the shear strength). A method to assess the critical cavity depth has been recommended by recent studies of Hossain et al. (2005; 2009a), which is incorporated into the new ISO guidelines (ISO 2012).

Figure 2. Two idealised scenarios after spudcan penetration in clay

For the site specific assessment of jack-ups in soft clay, the deep penetration of the spudcan foundations and the associated backflow are important. It is commonly perceived that the backflow will increase the foundation's bearing capacity, especially the horizontal and moment capacities. This is indeed indicated in the work by Templeton et al. (2005), Templeton (2009) and Zhang et al. (2011, 2012a), though the soil disturbance caused by spudcan installation was neglected in these numerical studies. The backflow also potentially acts as a 'seal' on top of the spudcan, allowing suction to be developed, as observed in centrifuge experiments (Cassidy et al. 2004a; Purwana et al. 2005; Gaudin et al. 2011). Therefore, tensile loading of the jack-up leg is possible, though it is not currently considered in conventional site specific assessment. Furthermore, the backflow is likely to enhance the foundation stiffness, importantly in rotation. This is beneficial for reducing the stress level in

critical parts of the jack-up structure, such as the leg-hull connection, as illustrated in Santa Maria (1988). Identifying and quantifying these perceived benefits will have significant economical implications for the jack-up industry while at the same time enhancing safety through improved understanding.

This paper proposes a model for describing the load-displacement behavior of the spudcan in soft clay that accounts for the effects of backflow. The framework of the model is based on displacement-hardening plasticity theory. Previous models in the same family include Schotman (1989), Nova and Montrasio (1991); Dean et al. (1997); Gottardi et al. (1999); Martin and Houlsby (2001); Cassidy et al. (2002a); Houlsby and Cassidy (2002); Bienen et al. (2006); Knappett et al. (2012). Foundations described by force resultant models can be readily incorporated into 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 soil-structure analyses can be found, amongst others, in Thompson (1996), Williams et al.(1998), Martin and Houlsby (1999), Houlsby and Cassidy (2002), Cassidy et al. (2001; 2002c), Bienen and Cassidy (2006, 2009) and Vlahos et al. (2011).

This paper will first set out the components of proposed force resultant model. A brief comparison of this model with a previous model for spudcans without backflow is then provided. To demonstrate the implications of the proposed model, integrated jack-up structure analyses under quasi-static monotonic push-over load are performed and results are discussed.

2 Proposed force resultant model

Based on plasticity theory, the force resultant model includes four components:

- i) a yield surface written directly in the *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 sections below detail the individual model components.

2.1 Hardening law

The model assumes that the yield surface expands with the plastic vertical displacement w_p (Figure 3). As the yield surface will be expressed as a function of pure vertical capacity, an accurate prediction of the vertical capacity of the spudcan at a given depth is important.

Figure 3. Different stages of penetration and shear strength profile

Hossain and Randolph (2009a) proposed the following expression to estimate the critical cavity depth (H_c , as shown in Figure 3b):

$$\frac{H_c}{D} = S^{0.55} - \frac{S}{4} \quad (1)$$

where S is defined by:

$$S = \left(\frac{s_{um}}{\gamma' D} \right)^{\left(1 - \frac{k}{\gamma'} \right)} \quad (2)$$

where s_{um} is the shear strength at seabed surface, k is the strength gradient as illustrated in Figure 3c, and γ' is effective unit weight of the soil.

For a spudcan embedded shallower than the critical cavity depth, i.e. $w_p \leq H_c$, the net capacity (V_0 , provided by the undrained soil strength) and total capacity (V_{total_C}) of the footing to resist compressive external vertical load are:

$$V_0 = N_{cs} s_{u0} A \quad (3)$$

$$V_{total_C} = N_{cs} s_{u0} A + \gamma' Vol_B + \gamma' w_p A \quad (4)$$

where N_{cs} is the shallow bearing capacity factor, s_{u0} is the undrained shear strength at the current plastic embedment depth (Figure 3c), A is the bearing area of the footing and Vol_B is the volume of soil displaced by the embedded part of the spudcan (Figure 3a).

As the spudcan penetrates past the critical cavity depth, soil backflow commences and the footing gradually becomes buried. With the onset of backflow, there is also a gradual transition from the general shear mechanism to the continuous flow-around mechanism (Hossain et al. 2005; Hossain and Randolph 2009a). In the current model, the transitional zone is assumed to be $0.5D$ (a smaller depth $0.3D$ is suggested by Hossain and Randolph (2009a); however, it is found a smoother load penetration response is obtained by assuming a larger transitional depth).

For a spudcan embedded beyond the transitional zone (i.e. $w_p \geq H_c + 0.5D$), the net and total

compressive vertical capacities of the spudcan are:

$$V_0 = N_{cd} s_{u0} A \quad (5)$$

$$V_{total_c} = N_{cd} s_{u0} A + \gamma' Vol_T \quad (6)$$

where N_{cd} is the deep bearing capacity factor and Vol_T is the total volume of the spudcan (Figure 3b). Note the cavity above the backflow does not contribute to the total capacity when the localised flow-around failure mechanism governs.

Due to the backflow, the spudcan is able to withstand tensile vertical load. The total capacity for a fully buried spudcan (i.e. $w_p \geq H_c + 0.5D$) to resist external tensile load is:

$$V_{total_T} = V_T - \gamma' Vol_T \quad (7)$$

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 . In this model, χ is assumed to be 0.6 based on centrifuge tests in Kaolin clay (Zhang et al. 2013a) when $w_p \geq H_c + 0.5D$. This ratio is consistent with other experimental research on extraction behaviour of spudcan footing in soft clay (Purwana et al. 2005; Bienen et al. 2009a and Gaudin et al. 2011).

For penetration depths in the transitional zone, i.e., $H_c < w_p < H_c + 0.5D$, a simple linear interpolation is proposed to calculate the net and total vertical compressive capacities. The tensile capacity is also expected to vary between zero and $0.6V_0$.

To calculate N_{cs} and N_{cd} respectively to be used in Eq. 3 to Eq. 6, the following equation is proposed following Hossain and Randolph (2009b):

$$N_{cs} \text{ or } N_{cd} = \left(\delta_{rem} + (1 - \delta_{rem}) e^{-3\xi_b/\xi_{95}} \right) N_{cs_ideal} \text{ or } N_{cd_ideal} \quad (8)$$

Eq. 8 corrects the bearing capacity factors in ideally non strain softening soil (soil sensitivity $S_t = 1$) for the effect of soil remolding during the spudcan penetration. In the equation, δ_{rem} is

the inverse of S_t . ξ_b represents the average cumulative shear strain as experienced by the soil particles as they traverse the flow mechanism around the spudcan and Hossain and Randolph (2009b) suggests ξ_b to be 2.4. ξ_{95} is equal to the cumulative strain required for 95% of full remolding and typical value is estimated in the range of 10 to 50 (Einav and Randolph 2005).

Hossain and Randolph (2009a) recommend N_{cs_ideal} to be determined by wished in place small strain finite element analyses with an open cavity. The numerical values are presented in their paper and are not repeated here for brevity. A simple equation is recommended to calculate N_{cd_ideal} :

$$N_{cd_ideal} = 10(1 + 0.065 \frac{w_p}{D}) \leq 11.3 \quad (9)$$

2.2 Yield surface

The following expression is proposed for the yield surface:

$$f = \left(\frac{H}{h_0 V_0} \right)^2 + \left(\frac{M/D}{m_0 V_0} \right)^2 - \frac{2eHM/D}{h_0 m_0 V_0^2} - \left(\frac{4}{(1+\chi)^2} \right)^2 \left(\frac{V}{V_0} + \chi \right)^2 \left(1 - \frac{V}{V_0} \right)^2 = 0 \quad (10)$$

where h_0 and m_0 define the peak ratios of H/V_0 and M/DV_0 in the in the VH ($M = 0$) and VM ($H = 0$) planes, the parameter e defines the eccentricity of the cross-section in the HM plane and the parameter χ is the tensile capacity ratio (V_T/V_0) as defined earlier. This expression is similar to that presented in Vlahos et al. (2008) and reduces to the form of Martin and Houlsby (2001), Houlsby and Cassidy (2002) and Cassidy et al. (2004b), amongst others, when no tensile capacity is assumed ($\chi=0$). Note that the yield surface is defined in terms of the net compressive vertical capacity V_0 rather than the total capacity.

As discussed above, the value of χ is taken to be 0.6 based on centrifuge testing results in normally consolidated Kaolin clay. However, the ratio may vary with soil type and

consolidation history. Changes are also expected to the size and shape of the yield surface as the failure mechanisms will change according to the soil. Therefore caution needs to be exercised when apply this model to soil conditions other than those investigated.

Table 1 lists the h_0 , m_0 and e values suggested for S_t in the range of 1 to 4 and w/D in the range of 0.5 to 3. Two-way linear interpolation is recommended for any combination of embedment depth and soil sensitivity that falls in the parametric range of this table. For $4 < S_t \leq 5$, the values for $S_t = 4$ are suggested, as numerical analyses reveal that the spudcan has approximately the same capacity in soil with $S_t = 5$ as $S_t = 4$ (Zhang et al. 2013c). These parameters are for spudcans fully submerged in backflow, and therefore should only be used when $w_p \geq H_c + 0.5D$.

The recommendation of the yield surface is based on the complementary studies of centrifuge experiments and numerical analyses. Employing a combined loading apparatus developed for a geotechnical drum centrifuge (Zhang et al. 2013a), the *VHM* bearing capacity surface of a typical spudcan was investigated at various embedment depths (up to $1.45D$) in normally consolidated kaolin clay in the enhanced gravity field of the centrifuge (Zhang et al. 2013b). To expand the experimental database for wider ranges of soil properties and embedment depths, a numerical approach with large deformation finite element (LDFE) simulation of the spudcan installation and subsequent small strain finite element analyses to define the *VHM* bearing capacity surface was proposed (Zhang et al. 2013c). In contrast to previous numerical analyses assuming a ‘wished in place’ spudcan (Zhang et al. 2011), the installation effects (large soil displacement and remoulding) were captured by combined use of LDFE and a strain softening soil model in simulation of spudcan penetration. The post installation shear strength profile is then used to calculate the combined *VHM* capacity surface. The size and

1 shape of the *VHM* bearing capacity surface for a spudcan in a normally consolidated soil was
2 defined for soil sensitivities up to five and embedment depths up to $3D$. The numerical results
3 with consideration of the spudcan installation effects (Zhang et al. 2013c) generally compare
4 well with the centrifuge experimental results. The yield surface expression (Eq. 10) together
5 with the size and shape parameters (Table 1) combines the experimental and numerical
6 findings in a slightly conservative manner.

7
8 It is noted that the yield surface parameters were derived immediately after installation (i.e.
9 no dissipation of excess pore pressures and subsequent consolidation was allowed for in the
10 numerical modelling and the experimental swipe tests were conducted immediately after
11 reaching the testing depth). The changes in the shear strength of the backflow soil and the soil
12 under the spudcan with time were not considered. This is considered a conservative approach.

Table 1. Yield surface parameters(for a buried spudcan: $w_p \geq H_c + 0.5D$).

S_t	w/D	h_0	m_0	e
1	0.5	0.186	0.090	0.374
	1	0.228	0.095	0.205
	1.5	0.277	0.102	0.063
	2	0.303	0.106	0.009
	3	0.325	0.107	-0.028
2.2	0.5	0.170	0.089	0.410
	1	0.206	0.092	0.280
	1.5	0.248	0.098	0.172
	2	0.267	0.101	0.096
	3	0.286	0.104	0.051
3	0.5	0.168	0.088	0.415
	1	0.199	0.090	0.310
	1.5	0.233	0.096	0.193
	2	0.255	0.099	0.137
	3	0.278	0.102	0.079
4	0.5	0.165	0.087	0.425
	1	0.193	0.088	0.332
	1.5	0.216	0.093	0.237
	2	0.236	0.096	0.182
	3	0.257	0.099	0.118

2.3 Elastic behavior

The elastic behavior is specified by a stiffness matrix that describes the relationship between load increment (δV , δH , δM) and elastic displacement increment (δw_e , δu_e , $\delta \theta_e$) as follows:

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

where K_V , K_H , K_M and K_C are dimensionless elastic stiffness coefficients, G is the shear modulus of the soil at the current w_p (Figure 3). For clay, the value of shear modulus G may

be taken as proportional to the undrained shear strength by $G = I_r s_{u0}$, where I_r is known as 'rigidity index'. The industry guidelines (SNAME 2008; ISO 2012) and Cassidy et al. (2002b) provide recommendations on selecting the appropriate I_r values.

The proposed model adopts the elastic stiffness coefficients reported by Zhang et al. (2012b) which apply to a spudcan shaped footing and consider the influence of increasing shear modulus with depth as well as the effect of backflow. It is noted that these elastic coefficients were derived assuming intact soil strength above and below the spudcan (i.e. installation effects were not considered). They, therefore, may be a slightly higher estimate. However, it is argued that this effect is minor compared to the uncertainty and sensitivity in the selection of the shear modulus G .

2.4 Flow rule

The plastic flow rule is primarily based on results from centrifuge experiments (Zhang et al. 2013b). The plastic potential has the following form:

$$g = \left(\frac{H}{\alpha_h h_0 V_0'} \right)^2 + \left(\frac{M/D}{\alpha_m m_o V_0'} \right)^2 - \frac{2eHM/D}{\alpha_h \alpha_m h_0 m_o V_0'^2} - \alpha_v^2 \beta_{34}^2 \left(\frac{V}{V_0'} + \chi \right)^{2\beta_3} \left(1 - \frac{V}{V_0'} \right)^{2\beta_4} = 0 \quad (13)$$

where V_0' is the dummy V_0 value for the plastic potential surface that intersects the yield surface at the current load point, and

$$\beta_{34} = \frac{(\beta_3 + \beta_4)^{(\beta_3 + \beta_4)}}{\beta_3^{\beta_3} \beta_4^{\beta_4} (1 + \chi)^{(\beta_3 + \beta_4)}}$$

$$a_h = 1$$

$$a_m = 1.35$$

$$a_v = 1.1$$

$$\beta_3 = \beta_4 = 0.45$$

3 Comparison with previous model for spudcan without backflow

Martin and Houlsby (2001) proposed a force resultant model based on a series of scaled 1 g model tests in heavily over consolidated clay (Martin and Houlsby 2000). It is colloquially known as ‘Model B’. Because of the soil strength profile and low level of in situ stress in those tests, no significant backflow was observed even at relatively deep embedment of 1.6 footing diameters. Therefore, the model’s yield surface only accounts for the contribution from the underside of the spudcan. For practical application, the model is most appropriate for spudcans in clay at depths where the no backflow scenario (Figure 2a) is predicted.

While the proposed model provides a consistent approach to predict the load-penetration response of spudcans in offshore soft clay, the model is intended for penetration depths beyond the transitional zone (i.e. with spudcans fully submerged in backflow) when it is used to assess the stability of the jack-up under environmental loading. At depths shallower than the predicted critical cavity depth, the model without backflow (Model B) is recommended. For depths in the transitional zone, Model B provides a conservative evaluation of the spudcan capacity. Definition of the transition zone requires further research to define the yield surface size and shape as well as the plastic potential.

In Table 2, a brief comparison between the proposed model of this paper and Model B is provided. In summary, the proposed model

- i) adopts a mechanism based approach to calculate the vertical capacity;
- ii) features a larger yield surface with the size parameters (h_0 and m_0) increasing with depth;
- iii) defines the yield surface at tensile vertical load; and

-
- 1 iv) uses higher elastic stiffness coefficient that account for the effect of backflow, the
2 increasing shear modulus with depth and a representative spudcan shape.
- 3 In addition, the proposed model has the flexibility to account for soil sensitivities $S_t = 1.0$ to
4 5.0 explicitly.

Table 2. Comparison of the proposed model with Model B

	Model B – without backflow	This model – spudcan fully buried in backflow
Hardening law	Bearing capacity factors evaluated by the method of characteristics for wished-in-place conical footings. No backflow General shear (shallow) mechanism	Based on centrifuge experiments and LDFE simulations. The soil sensitivity can be explicitly considered. Onset of backflow is assessed. Different mechanisms at various stages of penetration
Yield surface	The yield surfaces of both models can be generalised as: $f = \left(\frac{H}{h_0 V_0} \right)^2 + \left(\frac{M/D}{m_0 V_0} \right)^2 - \frac{2eHM/D}{h_0 m_0 V_0^2} - \beta_{12}^2 \left(\frac{V}{V_0} + \chi \right)^{2\beta_1} \left(1 - \frac{V}{V_0} \right)^{2\beta_2} = 0, \text{ where } \beta_{12} = \frac{(\beta_1 + \beta_2)^{(\beta_1 + \beta_2)}}{\beta_1^{\beta_1} \beta_2^{\beta_2} (1 + \chi)^{(\beta_1 + \beta_2)}}$	
	Only accounts for the underside of the spudcan. $\chi = 0$, i.e. tensile vertical load is not allowed, as a result, the combined bearing capacity diminishes to zero as vertical load approaches zero. $\beta_1 = 0.764$, $\beta_2 = 0.882$ and e varies with vertical load level. $h_0 = 0.127$, $m_0 = 0.083$, and are independent of embedment depth.	The contribution of backflow is considered. $\chi = 0.6$, i.e. tensile vertical load is allowed, which implies significant combined bearing capacity at low level of and tensile vertical load. This has been based on experimental evidence of kaolin clay in a geotechnical centrifuge (Zhang et al. 2013b), though caution should be taken to the level of tensile capacity provided. $\beta_1 = 1.0$, $\beta_2 = 1.0$ and e is independent of vertical load level. h_0 and m_0 values are listed in Table 1, and increase with embedment depth.
Elastic stiffness	Stiffness coefficients derived for: homogeneous soil profile conical footings no backflow	Stiffness coefficients derived for: linearly increasing soil strength profile one typical spudcan shaped footing different degrees of backflow
Flow rule	Associated in HM plane Plastic vertical displacement is corrected from association by a single parameter.	Slightly non-associated in the HM plane A separate plastic potential surface is defined.

4 Numerical formulation

Following the notations of Martin and Houlsby (1999), let

$$\boldsymbol{\sigma} = \begin{pmatrix} V \\ H \\ M \end{pmatrix}, \quad \boldsymbol{\varepsilon} = \begin{pmatrix} w \\ u \\ \theta \end{pmatrix}, \quad \frac{\partial f}{\partial \boldsymbol{\sigma}} = \begin{pmatrix} \partial f / \partial V \\ \partial f / \partial H \\ \partial f / \partial M \end{pmatrix} \quad \text{and} \quad \frac{\partial g}{\partial \boldsymbol{\sigma}} = \begin{pmatrix} \partial g / \partial V \\ \partial g / \partial H \\ \partial g / \partial M \end{pmatrix}$$

If the load increment is entirely within the current yield surface, the elastic load-displacement relationship is

$$\delta \boldsymbol{\sigma} = \mathbf{K}_e \delta \boldsymbol{\varepsilon} \quad (14)$$

where $\mathbf{K}_e$ is the elastic stiffness matrix defined in Eq. 12.

During an elasto-plastic increment, the elasto-plastic load-displacement relationship is:

$$\delta \boldsymbol{\sigma} = \mathbf{K}_{ep} \delta \boldsymbol{\varepsilon} \quad (15)$$

where $\mathbf{K}_{ep}$ is the elasto-plastic stiffness matrix, which needs to be defined. The total displacement $\delta \boldsymbol{\varepsilon}$ consists of elastic displacement, plastic displacement and coupled displacement due to change of elastic stiffness, as represented respectively by the three parts of the right-hand-side of the following equation

$$\delta \boldsymbol{\varepsilon} = \mathbf{K}_e^{-1} \delta \boldsymbol{\sigma} + \Lambda \frac{\partial g}{\partial \boldsymbol{\sigma}} + \delta w_p \frac{d \mathbf{K}_e^{-1}}{d w_p} \boldsymbol{\sigma} \quad (16)$$

where Λ is a non-negative scalar which determines the magnitude of plastic displacement and $d \mathbf{K}_e^{-1} / d w_p$ describes the change of flexibility matrix (inverse of elastic stiffness matrix) with depth due to:

- i) variation of shear modulus G with depth. This is important for modeling a soil profile with increasing strength with depth;
- ii) variation of the dimensionless elastic stiffness coefficients K_V , K_H , K_M , K_C with depth; and

iii) variation of the diameter of the contact area before the spudcan is penetrated beyond the *LRP*.

In an elasto-plastic increment, the consistency condition requires that the load point remains on the yield surface (i.e. $\delta f = 0$), therefore:

$$\frac{\partial f}{\partial \boldsymbol{\sigma}} \delta \boldsymbol{\sigma} + \frac{\partial f}{\partial w_p} \delta w_p = 0 \quad (17)$$

where

$$\frac{\partial f}{\partial w_p} = \frac{\partial f}{\partial V_0} \frac{dV_0}{dw_p} + \frac{\partial f}{\partial D} \frac{dD}{dw_p} + \frac{\partial f}{\partial h_0} \frac{dh_0}{dw_p} + \frac{\partial f}{\partial m_0} \frac{dm_0}{dw_p} + \frac{\partial f}{\partial e} \frac{de}{dw_p} \quad (18)$$

Note in both Eq. 16 and 17, δw_p can be replaced by $\Lambda(\partial g / \partial V)$ according to plasticity theory.

Based on Eq. 16 and Eq. 17, as demonstrated by Martin and Houlsby (1999), the elasto-plastic stiffness matrix can be calculated as:

$$\mathbf{K}_{ep} = \mathbf{K}_e - \frac{\mathbf{K}_e \left(\frac{\partial g}{\partial \boldsymbol{\sigma}} + \frac{\partial g}{\partial V} \frac{d\mathbf{K}_e^{-1}}{dw_p} \boldsymbol{\sigma} \right) \frac{\partial f}{\partial \boldsymbol{\sigma}}^T \mathbf{K}_e}{-\frac{\partial f}{\partial w_p} \frac{\partial g}{\partial V} + \frac{\partial f}{\partial \boldsymbol{\sigma}}^T \mathbf{K}_e \left(\frac{\partial g}{\partial \boldsymbol{\sigma}} + \frac{\partial g}{\partial V} \frac{d\mathbf{K}_e^{-1}}{dw_p} \boldsymbol{\sigma} \right)} \quad (19)$$

The footing model is implemented in FORTRAN. For any increment of load, displacement or combination of both, the resulting displacements/loads can be evaluated. The solution control follows the description of Martin and Houlsby (2001) and is not repeated here.

5 Jack-up quasi-static push-over analyses

In order to demonstrate the implications of the proposed model for site specific assessment of jack-up rigs in soft clay, quasi-static push-over analyses of a modern jack-up in an offshore soft clayey seabed were performed with the finite element program developed by Bienen and Cassidy (2006). Although Model B is most suitable for the ‘no backflow’ condition, results of comparative analyses are used to illustrate the effect of consideration of backflow in a jack-up push-over. For the purpose of this work, the jack-up is simplified as a plane frame and the environmental push-over load is assumed to act along the axis of symmetry of the structure at a position on the legs 15 m below the hull (Figure 4, this could be considered a mean water level, though no account for the water was made). The structure of the jack-up is represented by equivalent beam elements whilst the spudcans are represented by the force-resultant footing macro elements (the proposed model or Model B). The leg-hull connection is assumed to be rigid. Only elasticity of the jack-up structure is considered, limiting the failure of the system to the foundation. Geometric non-linearity (including the p - Δ effect) is included. Although the jack-up is deeply embedded, the interaction between the leg and the soil is not included in the analyses. In this work, only the one leg windward, two legs leeward orientation was considered.

In Table 3, the parameters for the jack-up structure, spudcan foundations and the seabed soil are tabulated. Two operation water depths of 20.6 m and 80.6 m are considered, representing a ‘short’ case and a ‘tall’ case, respectively (Figure 4). The total preload level is assumed to be 180 MN for both depths (this is twice the operational vertical load). A normally consolidated soil profile is considered with a small s_{um} of 1 kPa and a strength gradient of 1.5 kPa/m.

Figure 4. Illustration of two analysis cases: a short jack-up and a tall jack-up.

Table 3. The jack-up, spudcan and soil properties for the analyses

Jack-up and spudcan properties	
Transverse leg spacing	43.28 (m)
Longitudinal leg spacing	39.32 (m)
Total leg length below hull	60 (m) for the short jack-up 120 (m) for the tall jack-up
Water hull clearance	15 (m)
Equivalent leg area, A_{leg}	1 (m ²)
Second moment of area of the leg, I_{leg}	7.2 (m ⁴)
Equivalent area of the hull, A_{hull}	20 (m ²)
Second moment of area of the hull, I_{hull}	72 (m ⁴)
Young's modulus, E	200 (GPa)
Shear modulus, G_{steel}	80 (GPa)
Diameter of spudcan	14 (m)
Total operational vertical load	90 (MN)
Preload per spudcan	60 (MN)
Total preload	180 (MN)
Soil properties	
Seabed surface shear strength s_{um}	1.0 (kPa) (see Figure 3c)
Shear strength increase per meter, k	1.5 (kPa/m) (see Figure 3c)
Sensitivity, S_t	2.0
Ductility, ξ_{95}	30
Effective unit weight, γ'	6.85 (kN/m ³)
Rigidity index, $I_r = G/s_{u0}$	100

5.1 Prediction of vertical load penetration response

Figure 5 illustrates the load-penetration response predicted by the proposed model. At a preload level of 60 MN per leg, the predicted plastic embedment depth of the spudcan is 24.34 m. The net vertical resistance at the final embedment is 57.39 MN, with the difference from the total preload due to the buoyancy force. The back-calculated bearing capacity factor ($V_0/A s_{u0}$) is also presented in the figure. Its value is high at shallow embedment depths due to the large local soil heterogeneity (kD/s_{u0}) and decreases quickly with depth as kD/s_{u0} reduces.

However, once the spudcan penetrates past the transitional zone, the bearing capacity factor starts increasing steadily with depth. This pattern resembles the behaviour observed in the centrifuge experiments and large deformation finite element simulations (Zhang et al. 2013b; 2013c). It is necessary to comment on the importance of the transitional zone here. Before the fully localized flow-around failure mechanism is formed, the deep bearing capacity factor calculated by Eqs. 8 and 9 is not applicable. Otherwise, a sudden change of the bearing capacity factor (and in turn V_0) would be predicted, as shown in Figure 5 by the arrow. Furthermore, the influence of the cavity above the backflow on the total capacity diminishes gradually as the soil failure mechanism transitions. An immediate switch of bearing capacity equation from Eq. 4 to Eq. 6 at depth of H_c would result in unrealistic discontinuity of the total bearing capacity profile. Inclusion of the transitional zone provides a practical approach to predict the complete penetration profile.

Figure 5. Predicted load-penetration response by the proposed model

5.2 Comparison of predicted push-over responses with and without backflow

In order to compare the jack-up responses at the same spudcan embedment depth, the hardening law adopted in the proposed model is also used for the Model B analyses. After the preload, the vertical load on the jack-up is reduced to the operational vertical load, which was assumed to be 50% of preload for simplicity (at a total vertical embedment of 24.4 m). Horizontal push-over load is then applied to the jack-up and increased monotonically until failure is predicted.

5.2.1 Short jack-up response under a monotonic push-over load

Figure 6 shows the horizontal displacement at the *Hull Reference Point (HRP)*, Figure 4) of the jack-up. Overall, analyses with the two models predict similar jack-up responses. Upon applying the push-over load, the system response is elastic as the load state of all the spudcans falls inside of the yield surface established by preloading. The horizontal hull sway increases (nearly) linearly with the push-over load (with small non-linearities due to geometric effects in the structure). As the load state of a spudcan touches the initial yield surface, the system stiffness reduces. It reduces further when the remaining spudcan(s) yield. However, the proposed model accounting for backflow predicts a considerably higher ultimate push-over load than Model B, by approximately 50%. Besides that, the proposed model predicts higher system stiffness, reflected by the smaller hull sway under the same push-over load. This is mainly due to the higher spudcan elastic stiffness adopted by the proposed model, as compared in Table 2.

Figure 6. Horizontal hull response under a monotonic push-over load

Figure 7 illustrates the development of footing reactions with the push-over load H_{env} . As H_{env} is applied, the vertical reactions among the spudcans are re-distributed. Before yield of any of the spudcans, the vertical reaction of the single windward reduces with increasing push-over load and the same amount is equally transferred to the double leeward spudcans, maintaining the vertical equilibrium. This “push-pull” mechanism counters the majority of overturning moment imposed by the environmental load. During the elastic stage, the horizontal equilibrium is maintained by equally sharing the environmental load among the three spudcans. The moment reactions developed among the spudcans are also indistinguishable during this stage. When the leeward spudcans yield first as predicted by the proposed model, a larger portion of the horizontal load is taken by the single windward spudcan. The moment reaction on the leeward spudcans reduces quickly. Soon after the leeward spudcans, the windward spudcan also yields. The vertical reaction on the windward spudcan diminishes at an accelerated rate and enters the tensile regime toward failure of the system. In contrast to the prediction of this model, the analysis without backflow suggests the single windward spudcan will yield first and its share of the horizontal load reduces after yield, with a larger portion of the load undertaken by the leeward spudcans. The vertical

reaction of the windward spudcan remains compressive when approaching failure of the system. The reason for the predicted differences of footing responses between the two models is explained below.

Figure 7. Development of spudcan reactions under push-over load

Figure 8 compares the predicted load paths of the spudcan foundations during the push-over event in the VH , VM and HM planes. In Figure 8b, the VM envelope at $H = 0$ plane of both models are presented, illustrating the available bearing capacity for the spudcans before any push-over load is applied. Clearly, the proposed model suggests a significantly larger envelope than Model B. This is partially because tensile vertical load is allowed due to the backflow in this model whereas it is not the case in Model B. Furthermore, the m_0 value is larger in this model, as detailed in Table 2. Upon applying the push-over load, the H and M

loads on both the windward and leeward spudcans increase at similar ratios (Figure 8c and d). Due to the location of initial load state (operational vertical load only) relative to the yield envelope shown in Figure 8b (though the $H = 0$ envelope is not directly applicable as horizontal footing reaction is also mobilized), the proposed model predicts that the leeward spudcans yield first before the windward spudcan, whereas the opposite is the results from Model B. In Figure 8c and d, the HM cross-section of the yield surface at initial yield and failure of the system are presented for both models. Obviously the proposed model suggests considerable larger yield surface cross-sections on the HM planes, implying higher bearing capacity. As predicted by the proposed model, following yield the load paths of the leeward spudcans travel across the reducing HM cross-sections with increasing vertical load (though there is small amount of increase in w_p). The jack-up is predicted to fail through sliding of the leeward spudcans, and subsequently the windward spudcan. A similar mode failure (sliding) is predicted by Model B. Without backflow though, the windward spudcan will slide first as its load state approaches the horizontal peak of the HM cross-section before the leeward spudcans. This is reflected in the horizontal displacements of the spudcans as shown in Figure 9.

Figure 8. Load paths of spudcan foundations

Figure 9. Horizontal displacements of the spudcans during push-over event

5.2.2 Tall jack-up response under monotonic push-over load

Push-over analyses were also performed for a tall jack-up platform operating at a water depth of 80.6 m (Figure 4). The vertical preload and operational loads were kept the same as for the short jack-up. Figure 10 compares the hull responses between the tall and short jack-ups. Both models predict lower ultimate push-over load and larger hull sway (softer response) for the tall jack-up. This is due to the increased slenderness (height over width ratio) of the structure, which results in larger overturning moments imposed by the same amount of push-over load. Consistently, the proposed model predicts higher push-over capacity than Model B by about 26%. However, the relative effect of backflow is considerable smaller compared to the 50% difference predicted for the short jack-up.

Figure 10. Comparison of the horizontal hull responses between the short jack-up and the tall jack-up

Figure 11 compares the load paths of the spudcans experienced in the push-over event for both the short and tall jack-ups in the HM plane. The general pattern of the spudcan reactions is very similar in both cases. However, the initial slope of the load path before yield, i.e. the ratio of M/HD , is different. The tall jack-up experiences a much higher M/HD ratio, approximately three times that of the short jack-up, as shown in Figure 11. Along with the

spudcan load paths, the cross-sections of the initial yield surfaces at $V/V_0 = 0.25, 0.5$ of both models are shown. It is evident that for the same preload level, the yield surface of the proposed model is significantly larger in size than Model B, especially in the horizontal direction. The difference is comparatively smaller in the moment direction. When the load path of the spudcan approaches the yield surface at a higher M/HD ratio (as is the case for the tall jack-up), the available horizontal capacity for the spudcan foundations is lower, which results in a lower ultimate push-over capacity. This explains why the ultimate push-over capacity of the tall jack-up is less than that of the short jack-up. Moreover, the higher the M/HD ratio is, the smaller the difference between the yield surfaces of the proposed model and Model B, which explains why the predicted difference in ultimate push-over capacities between the two models is larger for the short jack-up than for the tall jack-up.

Figure 11. Comparison of the load paths between the tall and short jack-up analyses

The above compared the predicted jack-up response under a monotonic push-over load by the proposed model and Model B. Clearly, Model B, which neglects the effects of backflow, considerably under-predicts the push-over load of a jack-up platform with spudcans fully

submerged in soft clay. The magnitude of conservatism increases as the jack-up slenderness reduces. The proposed model reflects the different mechanism of a spudcan deeply penetrated in soft clay and will result in more economical site specific assessment results, without compromising safety.

5.3 Effect of soil sensitivity

In this section, a parametric study of the effect of soil sensitivity on the jack-up push-over response is performed. For this purpose, the tall jack-up case shown in Figure 4 is considered. Push-over analyses in soil with $S_t = 1, 2, 3, 4$ are performed. For consistency, the same preload level, operational water depth and water hull clearance are maintained for all cases.

Figure 12. Effect of soil sensitivity

As shown in Figure 12, as soil sensitivity increases, the embedment depth of the spudcan foundations increases under the same preload level. The ultimate push-over capacity, however, is shown to decrease with increasing soil sensitivity, though at relatively small magnitude. This is because of two compensating effects of h_0 and m_0 i) increasing with embedment depths for a specific S_t but ii) reducing with S_t for a specific embedment depth. Beside these two factors, the slenderness of the jack-up platform increases as the spudcans

penetrate deeper in more sensitive soil. The net effect is that the push-over capacity of the platform only reduces slightly with increasing soil sensitivity. However, this is expected to be influenced by other factors, such as the preload level, soil strength gradient (k) and ductility of the soil (ξ_{95}), which may alter the difference among the predicted embedment depths for different soil sensitivities and in turn influence the platform's push-over capacity.

6 Concluding remarks

A force-resultant footing model is herein proposed for spudcan foundations in soft clay based on a combined numerical and experimental study. Details of the experimental program providing verification of the model for predicting the load-displacement of a single spudcan can be found in Zhang et al. (2013b). The proposed model captures the influence of backflow and explicitly accounts for a range of soil sensitivities on the behavior of the spudcan.

Simple jack-up push-over analyses with fully buried spudcans in soft clay demonstrate considerable increases in the ultimate push-over capacity and system stiffness of the jack-up platform due to backflow. The proposed model removes excessive conservatism of current site specific assessment of jack-ups in soft clay without compromising safety.

Physical verification of the model's ability to predict the load-displacement behavior of a multi-footing jack-up during pushover should be pursued. Vlahos et al. (2008, 2011) provide verification of the original Model B footings by pushing a 1:250 scale three-legged jack-up to foundation failure. Following a similar methodology, but in this case for a 1:200 scale jack-up in a geotechnical centrifuge, Bienen et al. (2009b) and Bienen and Cassidy (2009) demonstrated the applicability of similar force-resultant models describing spudcan behavior in sand. Centrifuge tests are also required for verification of this proposed model in the

push-over analysis of jack-ups, to ensure flow around and burial of the spudcan footings. The 1:200 scale jack-up of Bienen et al. (2009b) could be used.

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