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Predicting Spud Can Extraction Resistance in Soft Clay

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ABSTRACT: Jack-ups are mobile offshore structures that are frequently relocated to new operation sites. To be relocated, the jack-up footings, known as spudcans need to be extracted from the seabed, using essentially the buoyancy of the hull as extraction force. This operation may be time consuming or even jeopardised if the spudcan extraction resistance is higher than the available extraction force. The maximum extraction (or breakout) resistance consists of suction at the spudcan base, weight of the soil above the spudcan, and soil shear resistance above the spudcan, with the contribution of the suction at the spudcan invert being the dominant component of the breakout resistance. This paper reviews an existing prediction method used to estimate spudcan extraction resistance and proposes an update of some of the input parameters based on insights obtained from a large database of experimental model data on two types of clays and for spudcan embedment up to three diameters.

KEYWORDS: Spudcan, Extraction resistance, Centrifuge modelling, Soft clay, Prediction method

1. INTRODUCTION

Self-elevating mobile jack-up units are the most common facilities used for offshore drilling operations in shallow waters, up to approximately 150 m depth (Figure 1). Once operation is completed, the jack-up is relocated to a new operation site, necessitating the jack-up footings, known as spudcans, to be extracted from the seabed. Difficulties in extraction can arise if the spudcans are deeply embedded in very soft clays. The development of high suction forces at the spudcan invert (Purwana et al., 2005; Gaudin et al., 2011) may augment the extraction resistance beyond the extraction force generated by the hull buoyancy, resulting in unexpected delays and additional costs.

As a part of assessing the jack-up removal process prior to going on a new location, an estimation of spudcan extraction resistance is therefore necessary for the jack-up operators to anticipate potential extraction issues and develop mitigation measures to facilitate spudcan extraction, such as water jetting for instance (Bienen et al., 2009; Gaudin et al., 2011).

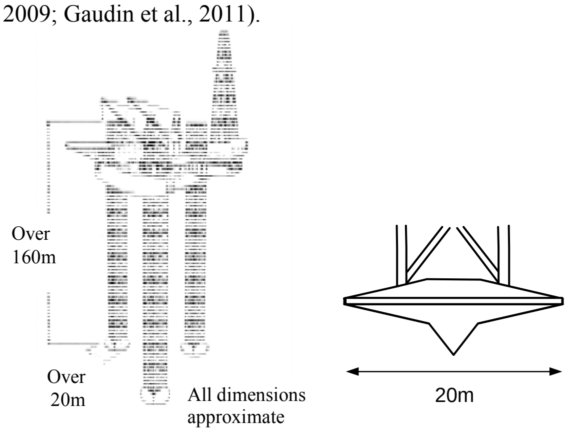

Figure 1 Typical jack-up and spudcan (modified after Reardon, 1986)

Two methods have been developed to estimate the maximum spudcan extraction resistance. They are detailed in Purwana et al. (2009) and Osborne et al. (2011), respectively. The method detailed in Purwana et al. (2009) is based on measurements of total and pore pressure at various locations on a model spudcan in centrifuge experiments for embedment up to 1.5 spudcan diameters, as well as information regarding soil failure mechanism from Particle Image Velocimetry analysis. The method contained in Osborne et al. (2011) is a modified version of this.

The objective of this paper is (i) to check the validity of the method established by Purwana et al. (2009) (called here after the reference method) for spudcan embedment up to 3 diameters and (ii) presents an update of some of the input parameters, based on insights obtained from an experimental model database of 24 centrifuge tests featuring spudcan extraction from normally consolidated clay.

2. DATABASE

The experimental database was gathered from data reported by Purwana et al. (2005), Purwana et al. (2009), Gaudin et al. (2011), Kohan et al. (2013a), Kohan et al. (2013b), and Kohan et al. (2014).

A total of 24 centrifuge test results were extracted and they are summarised in Table 1 in prototype scale. Scale factors for geometry, load, pressure, and the diffusion process can be found at Garnier et al. (2007) who made an inventory of the scaling laws and similitude questions relating to centrifuge modelling. Tests were conducted at 100 and 200 g, modelling spudcans of 6, 8, 12.5 and 17.1 m in diameter (30, 40, 85.56 and 125 mm in model scale).
For all tests, the test procedure consisted of three stages. In the first stage, spudcan penetration was performed in-flight in displacement or load control, under undrained conditions. The spudcan installation depths varied from 1 to 3 times the spudcan diameter. In the second stage, the jack-up operation period was simulated by maintaining a constant vertical load between 50% and 90% of the maximum installation load for up to five years in prototype scale, achieving varying degrees of consolidation in the soil around the

spudcan. It is noteworthy that the effect of operation load is less significant than that of operation duration (Purwana et al., 2005). Finally, in the third stage, spudcan extraction was performed in displacement control at a rate, v, resulting in a normalised velocity $V=vD/c_v$ greater than 30, where c_v is the virgin consolidation coefficient and D is spudcan diameter. This ensured that spudcan extraction was also performed under undrained conditions (Finnie and Randolph, 1994). The maximum extraction loads are reported in Table 1.

Centrifuge studies on spudcan extraction employed for assessment of the spudcan extraction resistance were performed in two different soils: UWA kaolin clay and Malaysian kaolin clay with a coefficient of consolidation c_v of approximately 2.8 to 4.8 m^2/year for UWA Kaolin clay and 40 m^2/year for Malaysian kaolin clay at a stress level consistent with the spudcan embedment. Soil characteristics including soil shear strength, soil unit weight, and soil effective stress at the spudcan installation depth for each centrifuge test are also provided in Table 1.

3. EXTRACTION FAILURE MECHANISM

The spudcan extraction failure mechanism was described in detail by Purwana et al. (2009) and Gaudin et al. (2011) for embedment ratios up to 1.5 times the spudcan diameter. The mechanism at peak extraction resistance is a combination of an uplift mechanism of the soil at the top of the spudcan, and a reverse end bearing at the spudcan invert associated with the development of negative excess pore pressure, namely suction (Figure 2). The main soil resistance is comprised of the weight of the soil above the spudcan, the resistance along a shear plane generated above the spudcan, and the suction pressure at the spudcan base.

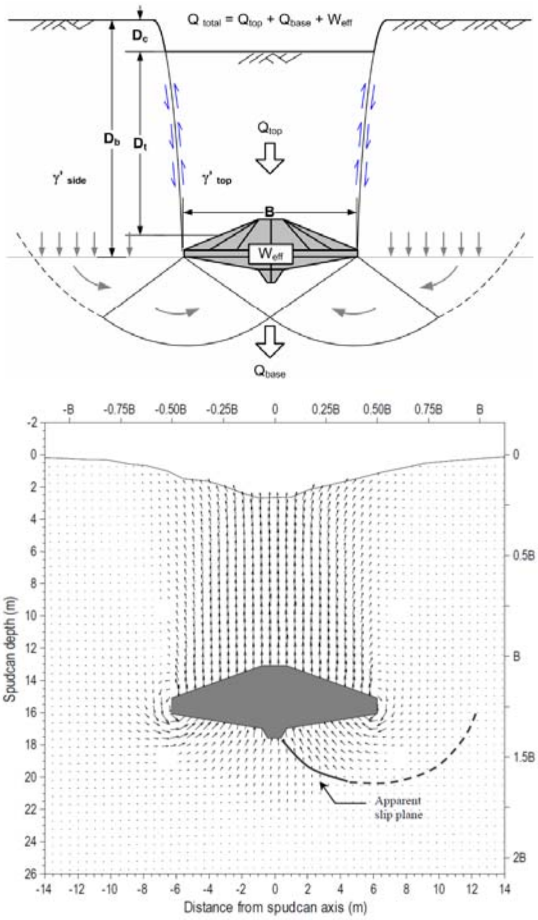

Figure 2 Observed spudcan breakout failure mechanism and diagram of breakout force components (after Purwana et al., 2009)

This has been identified by both PIV analysis (Purwana et al., 2006a) and numerical analysis (Zhou et al., 2009) of spudcan extraction in normally consolidated clay. Kohan et al. (2013b) demonstrated that this mechanism is also relevant for initial embedment ratio up to 3 times the spudcan diameter.

The components involved in the spudcan extraction resistance are influenced by the duration of the jack-up operation, i.e. by the degree of dissipation of excess pore pressures generated during installation, in the soil surrounding the spudcan. This results in the shear strength of the soil surrounding the spudcan increasing with operation time, and consequently, an increase in extraction resistance, as already demonstrated by Purwana et al. (2005).

It is noted that this mechanism may not apply for spudcans that have not seen any dissipation of excess pore pressures at immediate extraction. In this case, a reverse flow mechanism is more likely to develop.

4. EVALUATION OF THE REFERENCE METHOD

The method proposed by Purwana et al. (2009) (reference method) is based on the aforementioned breakout failure mechanism, identified using Particle Image Velocimetry (PIV) analysis for undrained extraction of a 12.5 m in diameter spudcan (prototype scale) from a depth of approximately 1.5 spudcan diameters in Malaysian kaolin clay (Purwana, 2006b). The vertical uplift force equilibrium condition assumed by Purwana et al. (2009) is illustrated in Figure 2.

The method has been presented in details in Purwana et al. (2009). It computes the uplift resistance as the sum of a resistance at the base Q_{base} (which accounts for overburden stresses), at the top Q_{top} and the submerged weight of the spudcan W_{eff}. Table 2 (see also Figure 3 and Figure 4) details the calculation of the first two components and summarises the parameters used in the method. When determining the net extraction resistance, W_{eff} is considered as zero.

To evaluate the performance of the method, the peak extraction resistance was calculated for each case, based on the input parameters reported in Table 3. Additional assumptions were made when data were missing, as explained below:

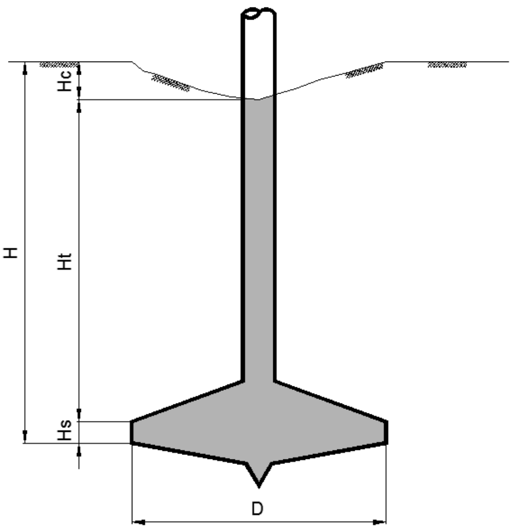

Figure 3 Variables defined in Table 2

i) To compute the top soil resistance, the height of the soil flowing back onto the top of the spudcan, which is a function of the depth of cavity formed during deep installation, needs to be assessed For cases where the cavity depth H_c was not reported, the solution developed by Hossain et al. (2006) was used as expressed in Table 2.

Table 1 Database

Test name	Reference	Ratio of the centrifugal acceleration to the earth gravity	Spudcan diameter D (m)	Spudcan depth ratio H/D (-)	Operation time t (day)	Operation load level of the maximum installation load V_{op}/V_p (-)	Breakout load Q_c (MN)	Soil unit weight □' (kN/m^3)	Soil shear strength at installation depth s_u (kPa)	Soil effect ive stress □'$_v$ (kPa)
3.0D2.0Y	Kohan et al. (2013b)	200	6.00	3.02	730	85%	-6.14	6.20	1.10 H	112.41
2.5D2.0Y	Kohan et al. (2013b)	200	6.00	2.50	730	85%	-5.62	6.20	1.10 H	93.06
2.0D2.0Y	Kohan et al. (2013b)	200	6.00	1.99	730	85%	-4.13	6.05	1.10 H	72.06
1.5D2.0Y	Kohan et al. (2013b)	200	6.00	1.48	730	85%	-3.29	6.05	1.10 H	53.54
1.5D3.0Y	Kohan et al. (2013b)	200	6.00	1.48	1095	85%	-3.43	6.05	1.10 H	53.66
1.5D1.0Y	Kohan et al. (2013b)	200	6.00	1.47	365	85%	-2.82	6.05	1.10 H	53.48
1.5D0.5Y	Kohan et al. (2013b)	200	6.00	1.47	183	85%	-2.42	6.05	1.10 H	53.48
1.5D0.0Y	Kohan et al. (2013b)	200	6.00	1.47	5	0%	-1.58	6.05	1.10 H	53.48
Nojet2	Kohan et al. (2014)	200	8.00	3.02	730	85%	-13.91	7.50	1.04 H	180.79
Nojet1	Kohan et al. (2013a)	200	8.00	2.50	730	85%	-14.24	7.50	1.08 H	182.01
S1UEnJ	Gaudin et al. (2011)	200	17.11	1.46	1664	90%	-80.52	6.00	1.17 H	150.00
S2UEnJ	Gaudin et al. (2011)	200	17.11	1.05	1664	90%	-46.97	6.00	1.17 H	108.00
GS1	Purwana et al. (2005)	100	12.50	1.45	<1	0%	-17.57	6.50	1.56 H	117.65
GS2	Purwana et al. (2005)	100	12.50	1.51	53	75%	-19.60	6.50	1.56 H	122.85
GS3	Purwana et al. (2005)	100	12.50	1.51	126	75%	-24.62	6.50	1.56 H	122.85
GS4	Purwana et al. (2005)	100	12.50	1.47	244	75%	-27.69	6.50	1.56 H	119.60
GS5	Purwana et al. (2005)	100	12.50	1.52	423	75%	-31.26	6.50	1.56 H	123.50
GS6	Purwana et al. (2005)	100	12.50	1.50	843	75%	-36.19	6.50	1.56 H	122.20
D-01	Purwana et al. (2009)	100	12.50	1.21	400	50%	-23.57	6.50	1 + 1.60 H	98.54
D-02	Purwana et al. (2009)	100	12.50	1.51	400	50%	-31.89	6.50	1 + 1.60 H	122.85
D-03	Purwana et al. (2009)	100	12.50	1.77	400	50%	-37.93	6.50	1 + 1.60 H	143.59
C-03	Purwana et al. (2009)	100	12.50	1.71	<1	0%	-20.12	6.50	1 + 1.60 H	138.65
C-02	Purwana et al. (2009)	100	12.50	1.51	<1	0%	-19.05	6.50	1 + 1.60 H	122.66
C-01	Purwana et al. (2009)	100	12.50	1.45	<1	0%	-17.39	6.50	1 + 1.60 H	117.59

Table 2 Parameters of reference method and method of this study

Parameter	Description	Reference method	Updated Method	Comments
D	Spudcan diameter (m)			
H	Spudcan installation depth (m)			
H_c	Cavity depth (m)	Measured during centrifuge test	$\frac{H_c}{D} = \left(\frac{s_{uHc}}{\gamma'.D}\right)^{0.55} - \frac{1}{4}\left(\frac{s_{uHc}}{\gamma'.D}\right)$	Provides universal method of all clay conditions
H_s	Spudcan side wall (m)			
H_t	Height of backfill above spudcan top surface (m)	See Figure 3		
γ'_{top}	Unit weight of soil at top (kN/m^3)	γ'_{top} =0.92 γ'	γ'_{top} =γ'	Simplification required without reliable method to estimate the change in γ'
γ'	Unit weight of soil (kN/m^3)	Soil property		
$s_{u,top}$	Average shear strength of backfill soil above spudcan after installation (kPa)			
$s_{u,base}$	Shear strength at the spudcan base level after installation (kPa)			
$f_{g,top}$	Change in shear strength of soil above spudcan top due to soil disturbance and any soil reconsolidation after spudcan installation (-)	0.67 for immediate extraction 0.87 for 400 days operation	Figure 8	Provide estimation for the full range of operational periods
$f_{g,base}$	Gain in shear strength of soil below spudcan base due to any soil reconsolidation after spudcan installation (-)	1.00 for immediate extraction 1.70 for 400 days operation	Figure 9	Provide estimation for the full range of operational periods
S	Shape factor	See Figure 4		
$N_{c,top}$	Breakout factor for top soil resistance[1] (-)	$if\ \frac{H_t}{D} > 1 \quad S*\left[2.56\ln\left(2\frac{H_t}{D}\right)\right] + \frac{\gamma'_{top}.H_t}{(s_{u,top})_{ave}\cdot f_{g,top}} \leq$ $if\ \frac{H_t}{D} < 1 \quad \left[3.5489*\left(\frac{H_t}{D}\right)\right] + \frac{\gamma'_{top}.H_t}{(s_{u,top})_{ave}\cdot f_{g,top}} \leq 1$		
$N_{c,base}$	Breakout factor for base soil resistance (-)	$N_{c,base} = 4.\left[1 + 0.2\left(\frac{H}{D}\right)\right] \leq 5.2$		
S_b	Adjustment factor for overburden stress at spudcan base level	$S_b = \begin{cases} 0.00 & for \quad \frac{H}{D} \leq 0.35 \\ 0.87\frac{H}{D} - 0.305 & for \quad 0.35 \leq \frac{H}{D} \leq 1.5 \\ 1.0 & for \quad 1.5 \leq \frac{H}{D} < 2.0 \end{cases}$	1.00 for all cases; therefore, is not part of the updated method	Simplification required without reliable method to estimate the change of S_b
Q_{uplift}	Total uplift resistance	$Q_{uplift} = (Q_{top} + Q_{base} + W_{eff})$		
Q_{top}	Top soil resistance	$Q_{top} = 0.25\pi D^2(N_{c,top}.s_{u,top})* f_{g,top}$		
Q_{base}	Base soil resistance[2]	$Q_{base} = 0.25\pi D^2(N_{c,base}.s_{u,base}.f_{g,base} - \gamma'.H.$	$Q_{base} = 0.25\pi D^2(f_{sr}.N_{c,base}.s_{u,base}\cdot f_{g,base}\cdot f_{ol} - \gamma'$ To consider effect of operation load and strength ratio	
W_{eff}	Submerged weight of spudcan	W_{eff} is ignored for net uplift resistance		
f_{ol}	Factor of operation ratio		$f_{ol} = 1 + 0.2(2.\frac{V_{op}}{V_p} - 1)$	$0.5 \leq V_{op}/V_p \leq 1$
f_{sr}	Factor of strength ratio		$f_{sr} = 1 + 0.4(\frac{\sigma'_v}{4s_u} - 1)$	Best fit to conditions of database

1. It should be noted that in calculation of breakout factor for top soil resistance, parameter $f_{g,top}$ was not mentioned in the original formulae presented by Purwana et al. (2009) and Purwana et al. (2010). However, the first author was informed by the personal correspondence that it is included in the overburden pressure term for determination of top soil resistance breakout factor (Purwana, 2010).

2. It is noted that for immediate or short consolidation periods $f_{g,base}$ will be 1 or close to 1 and Q_{base} can predict a negative value. This is inappropriate and reflects that the mechanism for this case is not realistic.

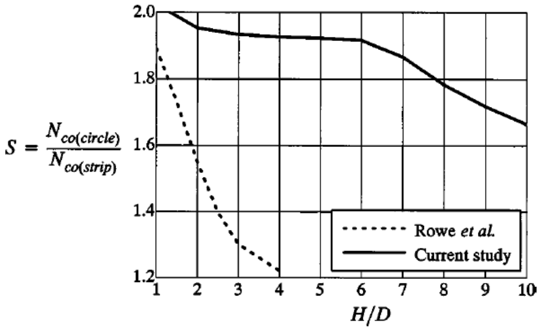

Figure 4 Shape factor (after Merifield et al., 2003)

$$\frac{H_c}{D} = \left(\frac{s_{uHc}}{\gamma'.D}\right)^{0.55} - \frac{1}{4}\left(\frac{s_{uHc}}{\gamma'.D}\right) \tag{1}$$

where H_c = Cavity depth (m); D = Spudcan diameter (m); γ' = Effective unit weight of soil (kN/m3); s_uH_c = Shear strength at the cavity depth (kPa).

ii) The gain in soil shear strength underneath the spudcan during operation time is characterised by the parameter $f_{g,base}$, which was evaluated as 1.00 and 1.70 by Purwana et al. (2009) (from numerical analysis) for immediate extraction and extraction after 400 days of operation, respectively. No values were reported for intermediate operational times. To evaluate the performance of the reference method for the entire database, presented in Table 1, $f_{g,base}$ was calculated for intermediate operation times, by linear interpolation between the degrees of consolidation achieved for 0 and 400 days. At 0 days operation time, it is logical to assume that the degree of consolidation is equal to 0. For 400 days and for Malaysian clay, the excess pore pressure dissipation during operation time was not reported in Purwana et al. (2009). Accordingly, data reported in Purwana et al. (2005) for test GS5 (423 days operation time) was used (Figure 5), leading to a degree of consolidation of 78% at 400 days. For tests in UWA kaolin clay, the degrees of consolidation were extracted from pore pressure measurements. Results of the linear interpolation of $f_{g,base}$ are listed in Table 3.

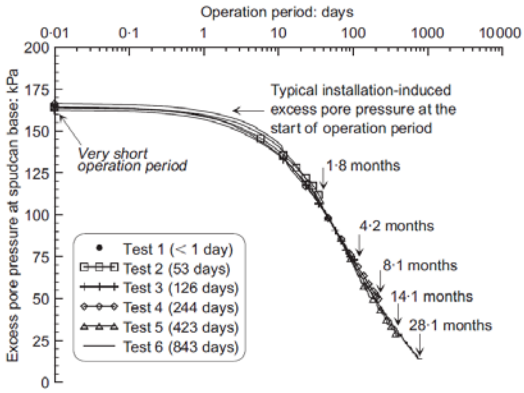

Figure 5 Dissipation of excess pore pressure at spudcan base during operation period (after Purwana et al., 2005)

iii) The change in soil shear strength at the top of the spudcan due to installation is characterised by the parameter $f_{g,top}$. Purwana et al. (2009) performed a series of T-bar tests in Malaysian kaolin clay to measure the shear strength of the remoulded soil at the spudcan top during the operation period. The shear strength was measured to reduce to 67% of the undisturbed shear strength immediately after spudcan installation, but increased by 30% (or 87% of the undisturbed soil shear strength) after 400 days reconsolidation period. This resulted in values of 0.67 and 0.87 recommended by Purwana et al. (2009) for 0 and 400 days of operation time, respectively. Similarly to the calculation of $f_{g,base}$, linear interpolation was conducted to assess $f_{g,top}$ for intermediate consolidation times.

From Figure 6, showing pore pressure responses at the end of the installation and operation time with respect to the hydrostatic pressure for test GS5, a degree of consolidation of 41% was deduced for 400 days of operation time. Calculated values of $f_{g,top}$ are presented in Table 3.

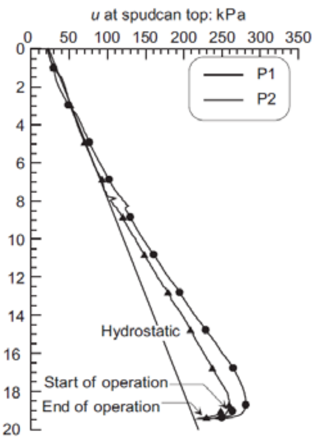

Figure 6 Pore pressure responses at spudcan top (after Purwana et al., 2005)

Predictions from the reference method are compared with the measured uplift resistances in Figure 7. Two observations are made:

1. The method predicts reasonably the peak extraction resistance in Malaysian clay (which is expected as the data underpin the development of this method), with a mean percentage error of about 9%. The performance is reduced for UWA kaolin clay, with a mean percentage error of about 57%. This potentially indicates that the performance of the reference method may be affected by the nature of the clay and that a better understanding of the various parameters associated with the soil characteristics is required.
2. The performance of the reference method is consistent for both clays for spudcan embedment up to 3 diameters, extending the validity of the method from embedment of 1.5 to 3 spudcan diameters. This is consistent with findings from Kohan et al. (2014), which demonstrated that the failure mechanism during extraction was identical between embedment of 1.5 and 3 spudcan diameters.

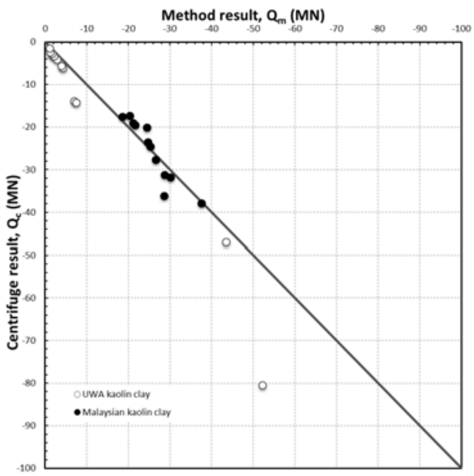

Figure 7 Predicted uplift force based on the reference method proposed by Purwana et al. (2009)

Table 3 Performance of the reference method

Test name	Cavity depth	Soil shear strength at top	Soil shear strength at base	Shear strength gain at top	Shear strength gain at base	Bearing factor at top	Bearing factor at base	Top resistance	Base resistance	Predicted breakout	Measured breakout	Error
-	D_c (m)	$s_{u,top}$ (kN/m^2)	$s_{u,base}$ (kN/m^2)	$f_{g,top}$ (-)	$f_{g,base}$ (-)	$N_{c,top}$ (-)	$N_{c,base}$ (-)	Q_{top} (MN)	Q_{base} (MN)	$Q_{breakout}$ (MN)	Q_c (MN)	- (%)
3.0D2.0Y	0.43	9.16	19.94	0.92	1.45	12.56	5.20	-3.25	-1.07	-4.32	-6.14	-42.05
2.5D2.0Y	0.43	8.03	16.51	0.97	1.54	12.56	5.20	-2.85	-1.12	-3.97	-5.62	-41.56
2.0D2.0Y	0.43	6.09	13.10	0.93	1.47	12.56	5.20	-2.16	-0.79	-2.95	-4.13	-39.91
1.5D2.0Y	0.43	4.25	9.74	0.87	1.36	12.56	5.18	-1.51	-0.46	-1.97	-3.29	-67.33
1.5D3.0Y	0.43	4.50	9.76	0.92	1.45	12.56	5.18	-1.60	-0.58	-2.18	-3.43	-57.27
1.5D1.0Y	0.43	3.97	9.72	0.81	1.26	12.56	5.18	-1.41	-0.31	-1.72	-2.82	-63.78
1.5D0.5Y	0.43	3.50	9.72	0.72	1.08	12.56	5.18	-1.24	-0.07	-1.31	-2.42	-85.23
1.5D0.0Y	0.43	3.27	9.72	0.67	1.00	12.56	5.18	-1.16	0.05	-1.11	-1.58	-42.56
Nojet2	0.46	11.04	25.14	0.89	1.39	12.56	5.20	-6.97	-0.06	-7.04	-13.91	-97.78
Nojet1	0.46	11.46	26.21	1.01	1.62	12.56	5.20	-7.23	-0.34	-7.58	-14.24	-87.90
S1UEnJ	2.09	13.29	29.25	0.87	1.36	12.56	5.17	-38.38	-13.80	-52.18	-80.52	-54.32
S2UEnJ	2.21	9.80	21.06	0.87	1.36	11.53	4.84	-26.92	-16.63	-43.55	-46.97	-7.86
GS1	1.92	9.42	28.24	0.67	1.00	12.56	5.16	-14.51	-4.09	-18.60	-17.57	5.56
GS2	1.92	10.55	29.48	0.72	1.09	12.56	5.20	-16.27	-5.41	-21.68	-19.60	9.59
GS3	1.92	11.63	29.48	0.79	1.22	12.19	5.20	-17.41	-7.92	-25.32	-24.62	2.78
GS4	1.92	12.03	28.70	0.84	1.31	11.52	5.18	-17.00	-9.59	-26.59	-27.69	-4.12
GS5	1.92	12.84	29.64	0.87	1.36	11.55	5.20	-18.19	-10.61	-28.80	-31.26	-8.53
GS6	1.92	12.79	29.33	0.88	1.37	11.41	5.20	-17.91	-10.70	-28.61	-36.19	-26.48
D-01	2.50	11.77	25.26	0.87	1.36	8.86	4.97	-12.79	-11.92	-24.71	-23.57	4.61
D-02	2.50	14.37	31.24	0.87	1.36	10.27	5.20	-18.12	-12.08	-30.20	-31.89	-5.58
D-03	2.50	16.59	36.34	0.87	1.36	11.60	5.20	-23.62	-13.98	-37.60	-37.93	-0.89
C-03	2.50	12.37	35.13	0.67	1.00	12.56	5.20	-19.07	-5.40	-24.47	-20.12	17.78
C-02	2.50	11.05	31.19	0.67	1.00	12.06	5.20	-16.35	-4.85	-21.20	-19.05	10.16
C-01	2.50	10.63	29.94	0.67	1.00	11.65	5.16	-15.20	-5.19	-20.38	-17.39	14.69

5. UPDATING THE INPUT PARAMETERS

The reference method is based on a rigorous description of the failure mechanism, incorporating the change in strength at the base and top of the spudcan resulting from installation and operation. They are estimated via two empirical factors $f_{g,base}$ and $f_{g,top}$, with limited insights into the values to adopt for intermediate operational times (i.e. between no and full consolidation) and different type of clays. The lower performance of the method for kaolin clay indicates that some aspects of the soil characteristics, which are not accounted for in the method, require a closer examination. Potential candidates include soil sensitivity, undrained bearing capacity factor, operation load, and consolidation coefficient.

The database gathered enables additional insights into each parameters involved in the reference method, although it is noted that a range of parameter combinations can be derived to fit individual test data (non-unique solution) and a holistic view of the fit to the database must be taken. Accordingly, the paper proposes updated recommendations to estimate the model parameters, notably the values of $f_{g,top}$ and $f_{g,base}$ as a function of the operation time and the type of clay used. Two plots are suggested that enable the assessment of gain in shear strength of soil at top and base of spudcan depending on the operation time. Additional recommendations relate to the estimation of the cavity depth, the unit weight of the soil on top of the spudcan, the overburden adjustment factor, and the introduction of two additional factors to account for the effects of the strength ratio and operation load. The updated recommendations for the input parameters are explained in detail below and are summarised in Table 2.

5.1 Cavity depth, H_c

The top soil resistance is a function of the height of the soil flowing back onto the top of the spudcan and of the depth of the cavity formed during installation. The solution developed by Hossain et al. (2006) is therefore recommended to estimate the cavity depth as explained in the previous section.

5.2 Unit weight of soil above the spudcan, γ'_{top}

The unit weight of the soil above the spudcan is slightly lower than that of the undisturbed soil due to the heavy remoulding occurring during penetration. Purwana et al. (2009) assumed the remoulded unit weight of Malaysian clay was about 92% of the virgin soil. Without indications about the variation of unit weight with the level of remoulding or estimated values for other types of clay, it is suggested for the updated method to use the virgin soil unit weight for all predictions. The impact of such a simplification on the performance of the method is limited, especially for shallow penetrations (and limited volume of soil). It is made for ease of calculation.

5.3 Change in soil shear strength above the spudcan, $f_{g,top}$

During installation, the soil is experiencing heavy remoulding and softening, resulting in a reduction of the shear strength of the soil resting above the spudcan. The reduction in shear strength is a function of the soil sensitivity S_t, which may vary between 2 to 2.5 for UWA kaolin clay and between 2 to 4 for the Malaysian clay. Regardless of the soil sensitivity, the remoulded soil regains some of its shear strength during operation through consolidation.

The evolution of the factor $f_{g,top}$ with operation time has been back calculated from the experimental data presented in Table 1. The process results in solving one linear equation with two unknowns, $f_{g,top}$ and $f_{g,base}$, requiring additional assumptions on both parameters. Accordingly, the following criteria were used to determine soil shear strength at top (and base as explained in next section) of the spudcan:

1. The lower bound of $f_{g,base}$ is 1, corresponding to the value immediately after extraction, before any consolidation occurs.

2. The upper bound of $f_{g,top}$ is equal to 1, corresponding to full strength recovery of the soil above the spudcan after full reconsolidation.

The values of $f_{g,top}$ that were considered a best-fit of the database are plotted in Figure 8 against the degree of consolidation U for all tests, excluding tests S1UEnJ, S1UEnJ, GS2 to GS4, and GS6, for which U is unknown. All points fall within in single logarithmic curve demonstrating a loss of strength of 40% immediately after installation and a rapid recovery to a value of about 78-85% of the initial shear strength after 10% of consolidation. This indicates that a fairly complex soil hardening process is taking place, which may involve mechanisms other than consolidation. For prediction purposes, it is suggested to adopt a value of $f_{g,top}$ of 0.6 for immediate extraction and a value of 0.78 to 0.85, increasing linearly between 10 and 100% consolidation. If more detailed knowledge of an offshore soil is known these limits may potentially be altered. However, without such knowledge the suggested values are a good guide that fits the experimental database well.

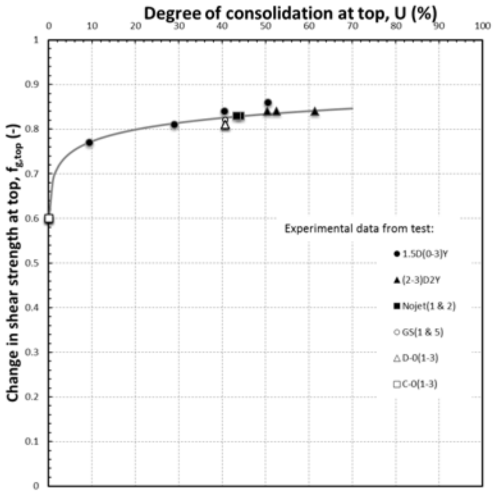

Figure 8 Change in shear strength at top of the spudcan during the operation time

5.4 Gain in soil shear strength underneath the spudcan, $f_{g,base}$

The soil below the spudcan consolidates under the load held during the operational period. This results in a gain in soil shear strength, described by the factor $f_{g,base}$, which lower bound value is established at 1. Values of $f_{g,base}$ considered to be the best holistic fit to the centrifuge data (and are consistent with the $f_{g,top}$ values of Figure 8) are plotted against the degree of consolidation U in Figure 9. This figure covers a wide range of degree of consolidation ensuing from different operation periods. It is evident from Figure 9 that a higher degrees of consolidation results in a larger gain in soil shear strength beneath the spudcan, with a linear fit reasonably representing the data. Values that range from 1 to 1.8 provide a reasonable fit for the two clays. The linear increase of strength with degree of consolidation is somewhat surprising and potentially indicates that the gain in strength is not homogenous underneath the spudcan.

5.5 Overburden pressure adjustment factor, S_b

The weight of the overlying soil imposes an overburden pressure at the spudcan installation depth. Since the failure mechanism at the spudcan invert has been identified as a reverse end bearing before changing to a localised flow around mechanism at the peak extraction resistance, the overburden pressure is required to be calculated to determine the net extraction resistance.

Purwana et al. (2009) considered the overburden stress as part of the base resistance, and assumed that it was partially mobilised from embedment between 0.35 and 1.5, before being fully mobilised for embedment ranging from 1.5 to 2 (see adjustment factor in Table 2). This factor was established from back calculation of the centrifuge data and therefore may be applicable only for Malaysian clay. In the present study, to cover all embedment, the two types of clay, and to simplify the approach, it is assumed that the overburden stress is fully mobilised at any spudcan embedment depth, resulting in an adjustment factor equal to one.

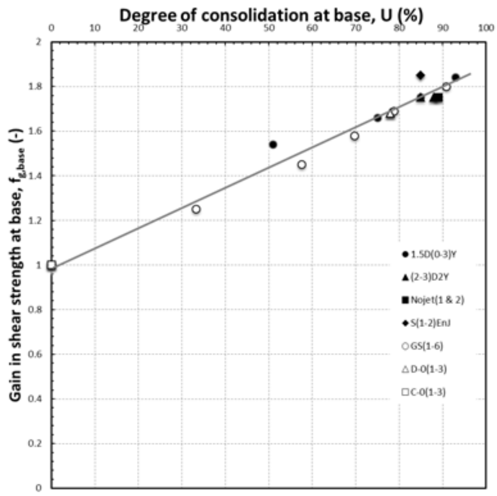

Figure 9 Gain in shear strength at base of the spudcan during the operation time

5.6 Effect of the operation load, f_{ol}

Purwana et al. (2005) examined the effect of the operational load on the spudcan extraction. Three tests with the operational load V_{op} set at 25%, 50% and 75% of the installation load V_p and with the same penetration depth and operation period were performed. Comparing the test results shows that the operation load does not influence the top soil resistance, whereas base soil resistance increases by approximately 10%, between an operation load ratio of 50% and 75% (Figure 10).

As the reference method was established based on the tests with an operation load ratio of 50%, a new factor f_{ol} (V_{op}/V_p) with a value of 1 at the operation load ratio of 0.5 and upper bound value of 1.2 at the operation load ratio of 1 is defined as:

$$f_{ol} = 1 + \alpha\,(2\,(V_{op}/V_p) - 1) \quad (2)$$

where V_{op} = Operation load (MN); V_p = Penetration load (MN); α = 0.2 (-). The value of α =0.2 has been chosen as it best fits on increase of 20% on the base soil resistance, as measured experimentally by Purwana et al. (2005). It should be noted that Equation 2 has been fit to the data for $0.25 \leq V_{op}/V_p \leq 1$. For V_{op}/V_p less than 0.5, a value of 0.9 is recommended.

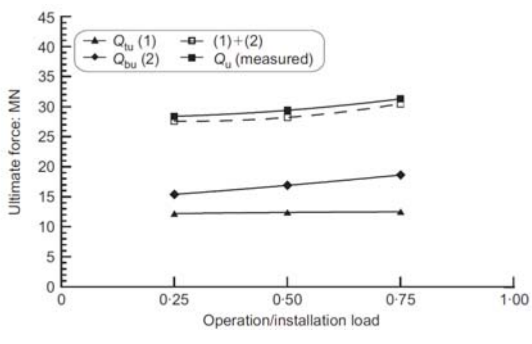

Figure 10 Variation of ultimate breakout forces for various ratios of operation load to installation load (after Purwana et al., 2005)

5.7 Effect of the strength ratio on the breakout factor for base soil resistance, f_{sr}

Figure 8 and Figure 9 demonstrate that a consistent set of parameters could be chosen for representing the consolidation characteristics and sensitivity of the two soils. However, the different performance of the two clays still requires differentiation and a parameter is required to explain the lower performance of the reference method for the UWA Kaolin clay.

One component of the increased extraction resistance measured in the UWA tests may be because of the difference in undrained shear strength profile; with the UWA tests having lower increasing strength with depth compared to the Malaysian clay tests. Usually for shallow foundations, considering their behaviour in compression, this would lead to a lower bearing capacity factor. However, close inspection of the lower bound bearing capacity factors of Houlsby and Martin (2003) for spudcans in an open cavity actually shows on increase in the bearing capacity factors with decreased strength gradient once the spudcan becomes embedded more than one diameter. As the mechanism of the reference method has a spudcan bottom contribution as the reverse of the Houlsby and Martin (2003) solution (i.e. uplift rather than penetration) it can be assumed that a lower strength gradient can increase the bearing capacity factor. This is accounted for by introducing a new factor f_{sr}.

An approach is proposed here whereas the effect of soil strength defined as the soil shear strength normalised by effective stress is considered. As the reference method was developed based on the results of the centrifuge tests in Malaysian clay, an additional factor f_{sr} (σ'_v / s_u) is defined as a function of the ratio of the effective stress normalised by soil shear strength for any soft soils to that of Malaysian kaolin clay. The effective stress normalised by soil shear strength for Malaysian kaolin clay is approximately 4; therefore, after performing a holistic fit of the database, f_{sr} can be expressed as below:

$$f_{sr} = 1 + \beta\,((\sigma'_v / s_u) / 4 - 1) \qquad (3)$$

where σ'_v = soil effective stress (kN/m^2); s_u = soil shear strength (kN/m^2); β = empirical factor = 0.4 (-). The value of the 0.4 provides the best fit to the experimental database.

5.8 Net extraction load, Q_{uplift}

The net extraction load Q_{uplift} is computed in the improved method as:

$$Q_{uplift} = Q_{top} + Q_{base} + W_{eff} \qquad (4)$$

$$Q_{top} = 0.25\,\pi D^2\,(N_{c,top}\,s_{u,top}\,f_{g,top})\ (MN) \qquad (5)$$

$$Q_{base} = 0.25\,\pi D^2\,(f_{sr}\,N_{c,base}\,s_{u,base}\,f_{g,base}\,f_{ol} - \gamma' H)\ (MN) \qquad (6)$$

6. DISCUSSION

Figure 11 illustrates the performance of the improved method through comparison between the predicted and experimental net extraction resistance. The method predicts the peak extraction resistance equally well in both types of clay (Figure 12), with a mean difference of about 8% (Table 4).

Although the method was used here to simulate test of immediate extraction (i.e. no load hold and therefore no excess pore pressure dissipation and consolidation), and was found to provide a resistance similar to the experiment, it is questioned if the mechanism that the reference method if based on is appropriate for this case. It is more likely a localised reverse flow mechanism for deep embedments and this mechanism should be the basis of a method to predict immediate extraction.

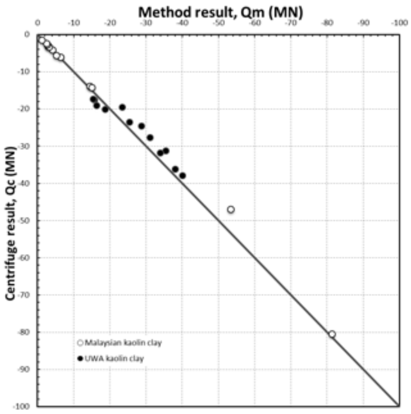

Figure 11 Results of improved prediction method

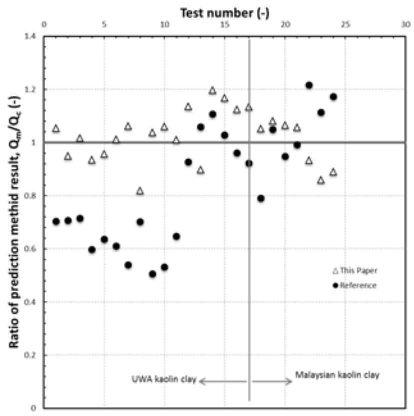

Figure 12 Comparing performance of reference method with updated formulation

Table 4 Predictions using updated parameters

Test name	Shear strength gain at top	Shear strength gain at base	Bearing factor at top	Bearing factor at base	Normalised strength	Factor of strength ratio	Factor of operation load	Top resistance	Base resistance	Predicted breakout	Error
-	$f_{g,top}$ (-)	$f_{g,base}$ (-)	$N_{c,top}$ (-)	$N_{c,base}$ (-)	$\square'_{v}/s_{u}$ (-)	f_{sr} (-)	f_{ol} (-)	Q_{top} (MN)	Q_{base} (MN)	$Q_{breakout}$ (MN)	- (%)
3.0D2.0Y	0.84	1.75	12.56	5.20	5.64	1.16	1.14	-2.84	-3.63	-6.47	5%
2.5D2.0Y	0.84	1.75	12.56	5.20	5.64	1.16	1.14	-2.33	-3.00	-5.33	-5%
2.0D2.0Y	0.84	1.75	12.56	5.20	5.50	1.15	1.14	-1.82	-2.38	-4.20	2%
1.5D2.0Y	0.84	1.75	12.56	5.20	5.50	1.15	1.14	-1.32	-1.76	-3.07	-7%
1.5D3.0Y	0.86	1.84	12.56	5.20	5.50	1.15	1.14	-1.35	-1.93	-3.28	-4%
1.5D1.0Y	0.81	1.66	12.56	5.20	5.50	1.15	1.14	-1.27	-1.59	-2.85	1%
1.5D0.5Y	0.77	1.54	12.56	5.20	5.50	1.15	1.14	-1.21	-1.36	-2.57	6%
1.5D0.0Y	0.60	1.00	12.56	5.20	5.50	1.15	1.14	-0.94	-0.35	-1.29	-18%
Nojet2	0.83	1.75	12.56	5.20	7.21	1.32	1.14	-6.26	-8.18	-14.44	4%
Nojet1	0.83	1.75	12.56	5.20	6.94	1.29	1.14	-6.54	-8.54	-15.09	6%
S1UEnJ	0.88	1.85	12.56	5.17	5.13	1.11	1.16	-32.79	-48.54	-81.33	1%
S2UEnJ	0.88	1.85	12.56	4.84	5.13	1.11	1.16	-22.20	-31.16	-53.36	14%
GS1	0.60	1.00	12.56	5.16	4.17	1.02	1.10	-10.23	-5.55	-15.78	-10%
GS2	0.68	1.25	12.56	5.20	4.17	1.02	1.10	-12.25	-11.23	-23.47	20%
GS3	0.74	1.45	12.56	5.20	4.17	1.02	1.10	-13.33	-15.43	-28.76	17%
GS4	0.78	1.58	12.56	5.18	4.17	1.02	1.10	-13.58	-17.55	-31.13	12%
GS5	0.82	1.69	12.56	5.20	4.17	1.02	1.10	-14.87	-20.59	-35.46	13%
GS6	0.86	1.80	12.56	5.20	4.17	1.02	1.10	-15.39	-22.68	-38.06	5%
D-01	0.81	1.68	12.45	4.97	3.90	0.99	1.00	-11.94	-13.53	-25.47	8%
D-02	0.81	1.68	12.56	5.20	3.93	0.99	1.00	-15.78	-18.19	-33.97	7%
D-03	0.81	1.68	12.56	5.20	3.95	1.00	1.00	-18.97	-21.15	-40.12	6%
C-03	0.60	1.00	12.56	5.20	3.95	0.99	1.00	-13.49	-5.28	-18.77	-7%
C-02	0.60	1.00	12.56	5.20	3.93	0.99	1.00	-11.67	-4.72	-16.38	-14%
C-01	0.60	1.00	12.56	5.16	3.93	0.99	1.00	-11.09	-4.38	-15.47	-11%

7. CONCLUSION

A database of centrifuge tests on spudcan extraction in two different types of clay has been gathered to assess the performance of an analytical method developed to predict the peak extraction resistance. The method proved to predict accurately the experimental results in Malaysian clay, but exhibited a significantly lower performance for UWA kaolin clay with a mean difference of about 57%.

A set of recommendations is proposed to update and improve the prediction method. The recommendations relates to the factors characterising the change in soil shear strength at the base and on the top of the spudcan and two new factors considering the effect of the operation load and strength ratio on spudcan extraction in clay. Additional details predicting when flow around occurs during installation have also been incorporated. The improved method demonstrated a higher degree of accuracy with a mean difference reduced to 8% for both types of clay.

8. ACKNOWLEDGMENTS

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