Perforation Drilling for Easing Spudcan-Footprint Interaction Issues

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ABSTRACT

The interaction between a spudcan and an existing footprint is a major concern during jack-up rig installation. No guidelines were provided in the recently finalised version of ISO guidelines 19905-1 in regards to mitigating spudcan-footprint interaction issues except some proposed considerations due to scarce of detailed investigation. This paper reports a measure for easing spudcan-footprint interaction issues, with the efficiency of perforation drilling tested through model tests carried out at 1g. The soil conditions tested simulate soft seabed strength profiles close to the mudline, varying the undrained shear strength. The most critical reinstallation locations of 0.5D and 1D (D = spudcan diameter) and existing footprint depth of 0.33D were investigated. The removal of soil (through perforation) inside the spudcan perimeter, with an area of 9% perforated, reduced the induced maximum moment, horizontal force, and horizontal sliding distance by up to 80%, 83%, and 76% respectively.

Keywords: Clays; Horizontal load; Moment; Perforation drilling; Sliding; Spudcan-footprint interaction.

1 INTRODUCTION

1.1 'Mobile' Jack-Up Rig and Spudcan-Footprint Interaction Issues

In the offshore oil and gas industry, 'mobile' independent-leg jack-up rigs are used widely in performing most offshore drilling in water depths up to around 150 m (CLAROM 1993; Randolph et al. 2005). The rigs are supported on 10 to 20-m diameter spudcan foundations. There has been a significant increase in the number of rig moves to the sites of the previously installed rig (or rigs) over the last 20 years. This is due to (a) increasing reliance on jack-up rigs for drilling and work-over activities for unmanned and subsea facilities (e.g. drilling additional wells or servicing existing wells); (b) increased jack-up tender assist operations (e.g. installing structures such as jackets, wind turbines); (c) expanding jack-up operational areas due to increased environmental capabilities; (d) reduced time spent at each location, giving increased opportunities for alternative units to operate over the individual field lives (Killalea 2002; Osborne and Paisley 2002; InSafeJIP 2010).

This has in turn caused one of the major concerns to the jack-up industry – interaction between a spudcan and an existing footprint during rig installation (see Figures 1 and 2), especially where the surface layer consists of clayey material. When a spudcan is located on or nearby an adjacent footprint slope, there is a tendency for the spudcan to slide towards the centre of the footprint, inducing excessive lateral forces and bending moments to the rig. Adverse spudcan displacement could result in an inability to install the jack-up in the required position, leg splay, structural damage to the leg, and at worst, bumping or collapsing into the neighbouring operating platform (ISO 2012).

The frequency of offshore incidents during installation near footprints has increased by a factor of four between the period 1979~1988 and 1996~2006 (Osborne 2005) and at an even higher rate over 2005~2012 (Jack et al. 2013). Examples of incidents include (a) leg damage

to the Mod V class Monitor and 101 jack-up platforms in the Central North Sea (Hunt and Marsh 2004); (b) an exceptionally high RPD reading in the Starboard leg of a jack-up rig due to sliding towards the footprint (Handidjaja et al. 2009); (c) some sliding was experienced by a rig installation as the legs were installed owing to the significantly different leg spacing from the previously installed L780-C Mod II unit (Brennan et al. 2006).

1.2 Spudcan Footprint Geometry

On removal of the jack-up unit, the legs are retracted from the seabed, leaving depressions (referred to as a crater or ‘footprint’) at the site. The depth and configuration of a footprint are a function of several factors. These include (a) the soil type, strength and stratification; (b) the spudcan final penetration depth before extraction; (c) the degree of soil backfill into the cavity above the spudcan during installation, as well as soil infill into the crater beneath the spudcan during and after removal of the spudcan; and (d) the spudcan shape. Over time, the geometry of the footprint may also be affected by the local sedimentary regime. Erbrich (2005), Brennan et al. (2006), Handidjaja et al. (2009) and Teh et al. (2010) reported four case histories, in offshore Australia and Southeast Asia, illustrating the footprints left by the extraction of spudcans. For instance, from bathymetry survey, Handidjaja et al. (2009) illustrated footprints of width $\kappa = x_F/D = 1.3$ (where D = spudcan diameter; see Figure 1 for definitions) and depth $\eta = z_F/D = 0.19$ in a layered soil with an undrained shear strength of $s_u = 20$ kPa at the mudline (see Figure 2a). The footprint craters consisted of a concave surface with a seabed gradient of about 1:3.5 along the slope. The spudcan foundations were 11.8 m in diameter and the penetration depths during preloading were 4.3~5.3 m. In a complex layered deposit, Teh et al. (2010) reported footprints of $\kappa = 1.03$ and $\eta = 0.24$ upon removal of the spudcans of $D = 14.6$ m (see Figure 2b).

The effect of initial soil strength on the configuration of footprint can be found from the results of half-spudcan tests carried out on clay deposits with various strength profiles (Hossain and Dong 2014). Figure 2c displays the images on soft, lightly over-consolidated clay ($s_u = 2.5 + 3.3z$ kPa). It was identified that the crater was mainly influenced by the depth of releasing suction, i.e. detaching the spudcan base from the underlying soil, during extraction. It is somewhat conical in soft clay, with relatively greater depth of sustained suction. During initial penetration of the spudcan, the soil flows towards the surface, which led to surface heave over a wider area around the periphery. This was coupled with the reverse backflow during extraction and dropping of backfilled soil around the periphery (and inside the crater), resulting in a soil hump around the periphery. The total width was 1.92~1.96D. These findings are consistent with Teh et al. (2010).

Clays soften as they are sheared and remoulded by a spudcan penetration-extraction sequence. The reduction in s_u decreases with the increase in radial distance from the footprint centre and along the depth. A zone of intensely remoulded soil was confined within 0.75D from the centre. However, the strength regains over the passing of time. After 1.5 years, the soil condition may found to be close to or stiffer than the undisturbed soil (Gan et al. 2012).

1.3 Previous Work and Existing Measures for Mitigating Spudcan-Footprint Interaction Issues

The influence of spudcan-footprint interaction has been addressed by a number of researchers (e.g. Gaudin et al. 2007; Leung et al. 2007; Cassidy et al. 2009; Gan et al. 2012; Kong et al. 2013). Most of these targeted understanding underlying behaviour. The critical offset ratio λ (defined as the ratio of the distance between the footprint centre and spudcan centre, b , and the spudcan diameter) was identified as 0.5~1 for inducing maximum lateral displacement of

~0.35D. From case histories, Handidjaja et al. (2009) found that if $\lambda > 1.5 \sim 1.7$, the effect of interaction can be neglected, while Teh et al. (2010) reported a minor slip for $\lambda = 1$.

However, attention to mitigate this issue is sparse. Several measures are suggested or considered for mitigation, including (a) infilling crater (Jardine et al. 2002), (b) capping the infilled crater with gravel loading platforms, (c) stomping (Jardine et al. 2002), (d) successive repositioning until the legs had stabilised in the desired plan position (Brennan et al. 2006), (e) water jetting along with the spudcan preloading (Handidjaja et al. 2009). In relation to case histories in the North Sea, Jardine et al. (2002) and Grammatikopoulou et al. (2007) examined the potential of using the former two. However, the results were mostly unsatisfactory. The pattern of soil movements became markedly asymmetrical (see Figure 3), which led to intolerable forces and moments developing in the jack-up leg before the target preload level was reached. Infilling the crater with different type of soil created additional problems such as punch-through. Gravel loading platform incurred the potential for slope or slope-base failure.

In addition, there is no concrete understanding of the mitigation measures and hence no corresponding guidelines in the recently finalised ISO guidelines 19905-1 (ISO 2012). This paper focuses on a mitigation measure of spudcan-footprint interaction issue.

An industry practice known as ‘perforation drilling’ is sometimes used to mitigate punch-through risk in layered clays, extracting soil from the upper strong layer before the jack-up is installed. Operations involve drilling numerous holes from the (yet to be installed) jack-up or a moored coring/drilling vessel (Handidjaja et al. 2004). A straight rotary flush drilling method is used with seawater as a drilling fluid to clean the hole and maintain stability. Two centrifugal pumps provide enough annular velocity to both maintain stability of the large diameter holes (~36 inch) being drilled and circulate clay cuttings out of the hole (Chan et al. 2008). The selection of hole size is governed by the available drilling equipment, with holes

typically drilled using a 26~36 inch mill tooth tricone bit. Occasionally, a 36~52 inch mill tooth hole opener is used to enlarge the perforations. The perforation drilling technique has been used recently to reinstall jack-ups without incident at several locations in Southeast Asia through layered clays (Maung and Ahmad 2000; Brennan et al. 2006; Kostelnik et al. 2007; Chan et al. 2008). Key normalised parameters from the perforation drilling operations were tabulated by Hossain et al. (2010) as drill diameter to spudcan (D) ratio $d_{\text{drill}}/D = 0.047\sim0.065$ (or effective hole diameter, $d_{\text{hole}}/D = 0.059\sim0.094$) and hole spacing (defined from centre to centre as s_{hole}) to spudcan diameter ratio, $s_{\text{hole}}/D = 0.099\sim0.15$. For these ratios the corresponding fraction of soil removed from inside the spudcan perimeter, $\eta_i = A_{\text{remi}}/A$ (where A_{remi} is the bearing area removed through perforation from inside the spudcan perimeter and A is the largest plan area of the spudcan), ranged from 13 to 60%.

Hossain et al. (2010, 2011) carried out a series model tests at 1g and 200g and proposed an optimised perforation drilling pattern. As it is impractical to drill a number of holes using a drill bit and drilling fluid on the laboratory floor or in a centrifuge mimicking a typical drilling operation in the field, a single operation drilling tool, referred to as the ‘shower head’, was developed for carrying out perforation operations effectively within a short period of time. It was showed that the removal of soil inside the spudcan perimeter, with an area of 9% perforated ($\eta_i = 9\%$ compared to 13~60% in the current practice), eliminated rapid leg run and severe punch-through on the two and four-layer seabed profiles tested. Recently, it has been emphasised repeatedly that perforation drilling may be useful in enabling a jack-up rig to be positioned on a location with existing footprint, mitigating interaction issue (e.g. Maung and Ahmad 2000). These were the motivation of this investigation using the identical drilling pattern.

1.4 Objective of Present Study

In an attempt to ease spudcan-footprint interaction issues, this paper reports results from an experimental study investigating the practicability of using perforation drilling. The soil conditions tested simulate soft seabed strength profiles close to the mudline. This was to be consistent with the previous studies exploring the influence of spudcan-footprint interaction (e.g. Gaudin et al. 2007; Leung et al. 2007; Cassidy et al. 2009; Gan et al. 2012; Kong et al. 2013). The footprint dimensions and spudcan reinstallation offset were considered accounting for the critical cases found from previous work and reported field data.

2 MODEL TESTS

2.1 Experimental Program

The experimental tests were performed at 1g on the laboratory floor. The tests were carried out in soil confined within a rectangular strongbox, which has (internal) plan dimensions of 650 × 390 mm and was 325 mm deep. An actuator was mounted on the strongbox allowing installation of the spudcan (and penetrometer) at a pre-determined rate. The experimental arrangement is shown in Figure 4. Three spudcan tests were undertaken on each sample (or box), including (i) spudcan test on level ground (SL); (ii) spudcan test adjacent to a footprint (SF); (iii) spudcan test on perforated site adjacent to a footprint (SFP) (see Table 1). A total of 15 tests were carried out on five boxes, as described in Table 1.

Spudcan-footprint interaction is a near soil surface (or mudline) problem. Under undrained loadings, the responses at shallow penetration depths depend primarily on cohesive soil strength (Martin and Houlsby 2000). This investigation is carried out at 1g with the aim of assessing the performance of perforation drilling at mitigating spudcan-footprint interaction

issues comparing relative spudcan responses adjacent to a perforated and un-perforated footprint. This investigation will be extended through centrifuge model tests and numerical analyses in the future.

2.2 Model Spudcan

Spudcan penetration tests were performed using a spudcan of 60 mm diameter (see Figure 3). The model was made from duraluminium and included a 13° shallow conical underside profile (included angle of 154°) and a 76° protruding spigot. The shape was chosen similar to the spudcans of the ‘Marathon LeTourneau Design, Class 82-SDC’ jack-up rig, as illustrated by Menzies and Roper (2008).

2.3 Instrumented Shaft and Sliding Device

The spudcan was rigidly connected to a model leg made from duraluminium, 16 mm in diameter and 235 mm long. The leg was instrumented by three sets of bending strain gauges and one set of axial strain gauges (see Figure 4) to record the vertical load applied and the bending moment generated during testing. For allowing horizontal movement to the leg, a special device was fabricated. The key feature of the device was the sliding displacement mechanism allowing free lateral displacement of the leg during the reinstallation. It consists of an immovable rectangular brushed aluminium bracket housing two highly polished non-friction aluminium shafts installed parallel to each other (see Figure 4). This configuration was used to ensure the leg connection underwent no rotation during the testing process. The leg itself was attached to an aluminium block into which the parallel shafts were threaded through. This restricted the block so that it would slide freely only on the horizontal plane. A linear displacement transducer (LDT) was connected to the sliding block to record the lateral displacement. The sliding block could also be ‘locked’ into place by four circular discs

positioned on either side, hence preventing any horizontal displacement. More details can be found in Gaudin et al. (2007).

In the field, the condition of a truss-work leg connected to the hull of a three-leg jack-up unit may be between these two – free sliding and rigidly fixed with no sliding. As such the tests on the last two boxes (B4~B5, Table 1) were carried out without using the sliding device.

2.4 Sample Preparation

Five series of tests (Box B1~B5, Table 1) were performed on soft specimens of kaolin clay as an abundance of reliable data regarding the geotechnical properties of kaolin clay is readily available (e.g. as given in Table 2). The soil conditions tested simulate soft seabed strength profiles close to the mudline, varying the undrained shear strength. It was found that undrained shear strengths may be no more than 2 to 15 kPa at seabed level and increase with depth at gradients typically in the range 1 to 2 kPa/m (Menzies and Roper 2008). In this study, soil deposit with a somewhat uniform strength profile was investigated.

A homogeneous slurry was prepared by mixing commercially available kaolin clay powder with water at 120% water content (twice the liquid limit) and subsequently de-airing it under a vacuum. The slurry was then carefully ladled into a strongbox and consolidated at 1g on the laboratory floor. Three different final consolidation pressures (50, 100 and 150 kPa) were used to achieve three different undrained shear strengths (see Table 1).

2.5 Soil Strength Determination

Soil characterisation tests were carried out using a T-bar penetrometer of diameter 5 mm and length 20 mm. On each box, strength assessments were undertaken immediately before starting spudcan penetration test and after completion of all spudcan tests, facilitating estimation of changes in undrained shear strength (s_u) profile over the testing time period.

Typical shear strength profiles are plotted in Figure 5, where z is the penetration depth at the T-bar mid-diameter and D the spudcan diameter. The measured T-bar resistance was interpreted using the analysis method suggested by White et al. (2010). This method accounts for changes in soil buoyancy and the reduced bearing factor mobilised during shallow embedment. First arbitrary values were considered for the depth of attaining a deep localised failure mechanism and corresponding deep bearing factor of $N_{T\text{-bar}}$. Profiles of initial shear strength and other parameters were calculated. Some iteration was then performed to obtain the actual shear strength profile. For the clay samples considered here, shallow embedment effects prevailed to $z = 135\sim 210$ mm i.e. up to the full penetration depth of < 120 mm (see also insert of Figure 5). The results show strength profiles with s_u reduced by about 1 kPa over a period of ~ 12 hours. This is because of swelling of the pre-consolidated soil sample and gradual absorption of ambient water. A reduction of strength of pre-consolidated soil samples was also reported by e.g. Hossain and Randolph (2010) and Hossain et al. (2010). At the conclusion of tests on each box, core samples were collected in order to determine the moisture content, giving average effective unit weight of $\gamma' = 7, 7.12$, and 7.25 kN/m³ for the samples pre-consolidated at 50, 100 and 150 kPa, respectively.

For interpretation of precise undrained shear strength profiles, s_u/σ'_v can be related to modified Cam clay parameters and over-consolidation ratio (OCR) following Roscoe and Burland (1968), Ladd et al. (1977), and Koutsotas and Ladd (1985) as

$$\frac{s_u}{\sigma'_v} = a \text{OCR}^b \quad (1)$$

where $\sigma'_v (= \gamma'z)$ is the vertical effective stress and $\text{OCR} = [(\text{pre-consolidation pressure of 50 or 100 or 150 kPa}) + \sigma'_v]/\sigma'_v$. The best fit of Equation 1 to the shear strength profiles plotted in Figure 5 was found with values for a and b of 0.32 and 0.79, respectively (see dashed lines

in Figure 5), which are consistent with the values ($a = 0.27$ and $b = 0.79$) can be obtained using the modified Cam clay parameters of the kaolin clay given in Table 2. Within the scope of this paper, the undrained shear strengths are indicative and used only for normalising maximum moment and horizontal force where a single value is required. As such, the strength profiles were idealised as approximately uniform over the full penetration depths, with a minor effect of softening close to the surface. Table 1 presents a summary of the idealised (and somewhat averaged) soil strengths. The sensitivity of the kaolin is 2.5~3.

The T-bar and spudcan were penetrated at a rate of 1 mm/s and 0.19 mm/s respectively, chosen to balance rate effects against ensuring undrained behaviour in clay. The normalised velocity index vD_e/c_v (where v is the penetration velocity, D_e the object effective diameter and c_v the consolidation coefficient for kaolin clay) was around 140, as recommended by Chung et al. (2006) and Low et al. (2008).

2.6 Formation of Footprint

Footprints were not created in a manner in which they usually form, i.e. through spudcan penetration followed by extraction, due to two reasons. First, as discussed previously, Gan et al. (2012) found that the remoulded shear strength will increase with the passing of time and it may be equal or even higher than the intact strength. Second, the tests were carried out at 1g and as such an impractical time frame (1:1) is required to simulate field situation. As such, craters were created manually on the surface of the consolidated samples.

A cutting tool comprised of a mounting frame and cutting blades was developed to create a footprint cavity (Figure 4). The frame was first mounted on the strongbox and the cutting blade was then rotated and cut into the soil forming a cavity of ideal conical shape until the desired footprint depth was reached. This eliminated the heterogeneity in undrained shear strength in a footprint created by spudcan penetration.

Accounting for Kong et al.'s (2010) findings and based on previous discussion on footprint geometry, craters dimensions were selected as 2D wide and 0.33D depth for relatively softer clay deposit (see Table 1). This is consistent and represents the upper bound of the profiles ("Footprint Type B", slope angle of 18.5°) presented in Figure 6.

2.7 Performing Perforation Drilling

On each box, a spudcan test was conducted adjacent to a footprint with the spudcan vicinity being perforated. The 'shower head' apparatus developed by Hossain et al. (2010, 2011) was modified for the 60 mm diameter spudcan used in this study (instead of $D = 40$ mm), and employed to perform the perforation drilling. The tool is shown in Figure 7. The water was pumped under a back pressure and was finally channelled through 64 brass tubes. These form a pattern of four circles in the narrow band of 0.9D to 1.2D. The circumferential and lateral spacing were selected to form an equilateral triangular grid with centre to centre spacing, $s_{\text{hole}}/D = 0.1$ and drill size relative to the spudcan diameter of 0.05. These various ratios fall within the practical range used offshore as discussed previously in Section 1.3. The length of the tubes below the plate was 1D ($h_{\text{hole}}/D = 1$).

In a test the shower head was attached to the actuator and positioned over the specific location of the specimen. Holes were created by simultaneously penetrating the tubes in the soil, and injecting water through them (somewhat simulating field drilling using fluid and not just coring). Consequently, the size of the holes was greater than that of the tubes. To circumvent excessive deterioration of the sample, the water flow was terminated once the showerhead reached its final penetration depth. A back pressure of 150 kPa along with the penetration rate of the shower head of 3 mm/s was used. Trial tests were conducted to optimise this back pressure and penetration rate. Examples of perforated footprints are shown in Figure 7. The diameter of the holes including washout was about $d_{\text{hole}}/D = 0.062$ (again falls within the

practical range of $d_{\text{hole}}/D = 0.059\sim 0.094$) and as such the fractional area removed inside the spudcan perimeter can be computed as 9%. While carrying out trial testing, the efficiency of the perforating holes was assessed through post-perforation bisection of the drilled site. No hole was found to be closed or collapsed, right down to the full penetration depth of the shower head, neither was closure or collapse of perforated holes observed, at least close to the surface, during execution of the spudcan test at the perforated site. No drilled hole collapse was also found to occur in centrifuge model tests (Hossain et al. 2011; who carried out drilling using the identical method and pattern), and in the field (Maung and Ahmad 2000; Brennan et al. 2006; Kostelnik et al. 2007; Chan et al. 2008).

2.8 Spudcan Penetration Test

On each soil sample (or box), two footprints were formed ($\kappa = 2$, $\eta = 0.33$, $\beta = 18.5^\circ$). To explore the critical cases, based on the previous discussion, two values of λ were selected (see Table 1). First spudcan test (SF) was conducted at an offset of $\lambda = 0.5D$ (i.e. full spudcan on the footprint slope; see Figure 1) or $1D$ (i.e. the spudcan tip on the crest of the slope, and hence half of the spudcan on the slope and the other half on the adjacent flat ground) from the footprint centre. For the second test (SFP), perforation drilling concentrically with the spudcan and up to 60 mm (i.e. $1D$) was followed by the spudcan penetration test. The final test was carried on a flat ground (SL).

3 PERFORMANCE OF PERFORATION DRILLING

3.1 Typical Response

The vertical penetration resistances are presented in terms of vertical load, V , as a function of spudcan tip penetration depth, d_{tip} . The readings from moment gauges were used to calculate

horizontal force, H_0 , and moment, M_0 , at the spudcan base level (referred to as load reference point, LRP, – see Figure 1). They are also plotted against d_{tip} . For the tests with free sliding (Box B1~B3), horizontal displacements, h , are plotted instead of H_0 . As each box consisted of three tests, one on an unperforated footprint, one after perforation, and one on a flat ground, all corresponding response profiles are included in each figure.

The results from tests on Box B1 are shown in Figure 8 (free sliding, $\lambda = 0.5$, $s_u = 11$ kPa; Table 1). A rapid sliding of $h = 24.8$ mm was recorded for the spudcan penetration adjacent to a footprint without perforation (Figure 8c). The corresponding maximum moment at LRP was $M_0 = 0.5$ N.m (Figure 8b). Perforation drilling reduced h by 76% and M_0 by 48%. This is because the perforated holes around the periphery of the advancing spudcan provided a preferential penetration path, forcing the spudcan to penetrate somewhat vertically. In addition, this allowed the soil to be compressed rather than flowing back to the surface. This prevented the tendency of sliding significantly. Figure 9 illustrates the associated mechanisms. There was a 51% reduction of vertical load resistance at the footprint toe level (at $d_{tip} = 20$ mm; see Figure 8a). As expected, the moment and sliding responses for the test on a flat ground (SL) were consistently negligible (e.g. see Figure 8) and hence not included in the discussion.

Typical results for rigidly fixed with no sliding condition are illustrated in Figure 10 (Box B4, $\lambda = 0.5$, $s_u = 6$ kPa; Table 1). For the spudcan penetration adjacent to an unperforated footprint, the maximum moment and horizontal force (above the footprint toe level along the penetration depth) were recorded as 0.12 N.m and 1.3 N (Figures 10b and c). A somewhat vertical spudcan penetration by means of perforation drilling has allowed for reducing the induced moment by 75% and horizontal force by 82%. Again the vertical load resistance was reduced by 49% (at $d_{tip} = 20$ mm; see Figure 10a).

Two points are worth noted. First, for penetration depths $\leq$ the footprint toe level (i.e. $d_{tip} \leq 20$ mm), as expected, the vertical load resistance on a flat ground was consistently higher compared to that on a sloped ground (Figures 8a and 10a) due to lower spudcan base-soil contact area (Kong et al. 2015; Zhang et al. 2015). The difference was more obvious for the free sliding penetration (see Figure 8a). Second, a somewhat S-shaped response profiles can be seen in Figures 10b and 10c for the spudcan penetration adjacent to an unperforated footprint (Test SF). This was perhaps partly because of the balancing of various mobilised failure mechanisms (as illustrated by Kong et al. 2015 and Zhang et al. 2015) and corresponding leg splay along the penetration depths of this rigidly fixed (allowing no sliding) spudcan, and partly because of the existence of a profound drop in the corresponding undrained shear strength profile around 80 mm depth (see Figure 5). Perforation drilling ensured a somewhat vertical penetration and hence eliminated the former, leading to negligible M_0 and H_0 along the penetration depth. The reduction around $d_{tip} = 80$ mm was due to the latter (i.e. the drop in the shear strength profile). The resultant was the crossed response profiles from Tests SF and SFP (see Figures 10b and 10c).

3.2 Performance of Perforation Drilling

For all tests with and without perforation drilling, a comparison in terms of the magnitude of maximum moment, M_0 , horizontal force, H_0 , or sliding distance, h , and maximum vertical load, V , are provided in Table 1. These are presented in Figure 11 in non-dimensional forms, M_0/As_uD , H_0/As_u and h/D , as a function of λ (as the other parameters relevant to footprint were constant as $\kappa = 2$, $\eta = 0.33$, $\beta = 18.5^\circ$). s_u values given in Table 1 were used for these normalisations. In general, the values decrease with increasing λ . This trend is consistent with Kong et al. (2013). All the critical parameters reduce significantly as a result of perforation

drilling, showing the potential of using perforation drilling in easing spudcan-footprint interaction issues.

4 CONCLUDING REMARKS

The severity of spudcan-footprint interaction issues is quantified by the following measures: induced moment and horizontal force at the spudcan base level and/or distance of the leg sliding towards the footprint toe. The experimental results presented in this paper have shown that perforation drilling can be effective at easing of spudcan-footprint interaction issues during jack-up installation adjacent to existing footprints. With test specimens simulating strength profiles of surface layers, footprint geometry and spudcan installation offset, that caused spudcan-footprint interaction issues to occur, perforating these profiles by only 9% of the total internal spudcan area concentrically with the installing spudcan consistently reduced the maximum moment, horizontal force and sliding distance by 30~80%, 70~82%, and 48~76% respectively. The penetration resistance was also reduced up to the depth of perforation, which may have insignificant influence in the field where spudcans penetrate 2~3 diameters in soft clay deposits.

The results are from preliminary investigations carried out at 1g. These will be fine-tuned through centrifuge model tests, and on stiff clay deposits. Tests will also be conducted to assess the performance of other potential mitigation measures such as stomping and successive repositioning, and the results will be compared with those achieved using perforation drilling.

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500 Nomenclature

501 A spudcan plan area at largest section

502 A_{remi} bearing area removed through perforation from inside spudcan perimeter

503 b offset distance

504 c_v coefficient of consolidation

505 D foundation diameter at largest section

506 D_e object (area equivalent) diameter507 d_{tip} penetration depth of spudcan tip508 d_{drill} drill bit or tube outer diameter509 d_{hole} hole diameter510 H_0 horizontal force at spudcan base level

511 h horizontal sliding distance

512 h_{hole} hole depth513 M_0 moment at spudcan base level514 S_{hole} hole spacing515 s_u undrained shear strength

516 V vertical force

517 v object penetration velocity

518 x_F footprint diameter

519	z	depth below soil surface
520	z_F	footprint depth
521	β	footprint angle
522	η	ratio between footprint depth and spudcan diameter = z_F/D
523	η_i	fraction of soil removed through perforation from inside the spudcan perimeter
524	σ'_v	vertical effective stress
525	γ'	soil effective unit weight
526	κ	ratio between footprint diameter and spudcan diameter = x_F/D
527	λ	ratio between offset distance and spudcan diameter = b/D
528		

529

Table 1. Summary of model tests conducted (D = 60 mm)

Box	Pre-Consolidation pressure (kPa)	Undrained shear strength, s_u (kPa)	Spudcan installation distance, $\lambda = b/D$	Horizontal displacement	Spudcan tests													
					M_0 (N.m)		H_0 (N)		h (mm)		V (N) (at footprint toe)		$M_0/As_u D$		H_0/As_u		h/D	
					SF	SFP	SF	SFP	SF	SFP	SF	SFP	SF	SFP	SF	SFP	SF	SFP
B1	100	11	0.5	Free	0.50	0.26	-	-	24.80	6.00	129.8	63.10	0.27	0.14	-	-	0.41	0.10
B2	150	16	1	Free	0.45	0.23	-	-	16.40	8.60	92.60	54.80	0.17	0.09	-	-	0.27	0.14
B3	150	16	0.5	Free	0.60	0.42	-	-	17.30	6.70	131.60	57.70	0.22	0.16	-	-	0.29	0.11
B4	50	6	0.5	Locked	0.12	0.03	1.30	0.23	-	-	115.50	59.10	0.12	0.03	0.077	0.014		
B5	100	11	1	Locked	0.10	0.02	0.60	0.18	-	-	101.10	56.30	0.05	0.01	0.019	0.006		

530 SF: spudcan test adjacent to a footprint

531 SFP: spudcan test on perforated site adjacent to a footprint

532 Footprint dimensions: Depth, $\eta = z_F/D = 0.33$; Width, $\kappa = x_F/D = 2$; Slope angle, $\beta = 18.5^\circ$

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535

536 **Table 2. Engineering properties of kaolin clay (data from Stewart 1992; Acosta-**537 **Martinez and Gourvenec 2006)**

Property	Value
Liquid limit, LL: %	61
Plastic limit, PL: %	27
Plasticity index, I_p : %	34
Specific gravity, G_s	2.6
Critical state frictional constant, M	0.92
Slope of normal consolidation line, λ	0.205
Slope of swelling line, κ	0.044
Parameter $\Lambda = (\lambda - \kappa)/\lambda$	0.785
Consolidation coefficient (mean), c_v : m ² /year	~2.6
Permeability coefficient (at 85 kPa), k_v (m/sec)	$1.3 \times 10^{-10} \sim 5.1 \times 10^{-9}$

538 No of Figure: 12

539 **Fig. 1** Schematic diagram of spudcan-footprint interaction and mitigating perforation drilling

540 **Fig. 2** Geometry of spudcan footprints: (a) Diagram of existing footprints ($\eta = z_F/D = 0.19$, κ
541 $= x_F/D = 1.3$) and arrangements of a rig emplacement over them (from a case history in
542 offshore Southeast Asia, after Handidjaja et al. 2009); (b) Diagram of existing footprints ($\eta =$
543 $z_F/D = 0.24$, $\kappa = x_F/D = 1.03$) and arrangements of a rig emplacement over them (from a case
544 history in offshore Southeast Asia, after Teh et al. 2010); (c) Footprints from centrifuge tests
545 (after Hossain and Dong 2014)

546 **Fig. 3** Measures for mitigating spudcan-footprint interaction (after Grammatikopoulou et al.
547 2007): (a) Vectors of incremental ground displacement – infilled crater; (b) Vectors of
548 incremental ground displacement – infilled crater capped with gravel loading platform

549 **Fig. 4** Experimental set-up for a spudcan test on a footprint

550 **Fig. 5** Shear strength profiles measured before and after spudcan tests

551 **Fig. 6** Comparison of footprint profiles record (after Kong et al. 2010)

552 **Fig. 7** Drilling tool shower head and perforated footprints

553 **Fig. 8** Results of tests on Box 1: free sliding leg ($\lambda = 0.5$, $\kappa = 2$, $\eta = 0.33$, $\beta = 18.5^\circ$; Table
554 1): (a) Load-penetration response; (b) Moment at spudcan base level; (c) Sliding distance

555 **Fig. 9** Effect of perforation spudcan sliding and soil flow mechanisms ($\lambda = 1$, $\kappa = 2$, $\eta =$
556 0.33 , $\beta = 18.5^\circ$; Box 2, Table 1): (a) Spudcan penetration adjacent to footprint: sliding with
557 significant soil flow to surface towards footprint toe; (b) Spudcan penetration on perforated
558 site adjacent to footprint: minimal sliding with little soil flow to surface

559 **Fig. 10** Results of tests on Box 4: fixed leg ($\lambda = 0.5$, $\kappa = 2$, $\eta = 0.33$, $\beta = 18.5^\circ$; Table 1): (a)
560 Load-penetration response; (b) Moment at spudcan base level; (c) Horizontal force at spudcan
561 base level

562 **Fig. 11** Effect of perforation: free sliding leg ($\kappa = 2$, $\eta = 0.33$, $\beta = 18.5^\circ$; Box 1~3, Table 1):
563 (a) Non-dimensional moment; (b) Normalised sliding

564 **Fig. 12** Effect of perforation: fixed leg ($\kappa = 2$, $\eta = 0.33$, $\beta = 18.5^\circ$; Box 4~5, Table 1): (a)
565 Non-dimensional moment; (b) Non-dimensional horizontal force

566

567

568

Figure 1. Schematic diagram of spudcan-footprint interaction and mitigating perforation drilling

2(a) Diagram of existing footprints ($\eta = z_F/D = 0.19$, $\kappa = x_F/D = 1.3$) and arrangements of a rig emplacement over them (from a case history in offshore Southeast Asia, after Handidjaja et al. 2009)

2(b) Diagram of existing footprints ($\eta = z_F/D = 0.24$, $\kappa = x_F/D = 1.03$) and arrangements of a rig emplacement over them (from a case history in offshore Southeast Asia, after Teh et al. 2010)

2(c) Footprints from centrifuge tests

Figure 2. Geometry of spudcan footprints

3(a) Vectors of incremental ground displacement – infilled crater

3(b) Vectors of incremental ground displacement – infilled crater capped with gravel loading platform

Figure 3. Measures for mitigating spudcan-footprint interaction (after Grammatikopoulou et al. 2007)

Figure 4. Experimental set-up for a spudcan test on a footprint

Figure 5. Undrained shear strength profiles measured before spudcan tests and after all spudcan tests on a box

Figure 6. Comparison of footprint profiles record (data from Kong et al. 2010)

Figure 7. Drilling tool shower head and perforated footprints

8(a) Load-penetration response

633

634 8(b) Moment at spudcan base level

8(c) Sliding distance

Figure 8. Results of tests on Box 1: free sliding leg ($\lambda = 0.5$, $\kappa = 2$, $\eta = 0.33$, $\beta = 18.5^\circ$;

Table 1)

644

645 9(a) Spudcan penetration adjacent to footprint: sliding with significant soil flow to surface
646 towards footprint toe

647

648 9(b) Spudcan penetration on perforated site adjacent to footprint: minimal sliding with little
649 soil flow to surface

650

651 **Figure 9. Effect of perforation spudcan sliding and soil flow mechanisms ($\lambda = 1$, $\kappa = 2$, η**
652 **$= 0.33$, $\beta = 18.5^\circ$; Box 2, Table 1)**

653

654 10(a) Load-penetration response

655

656

657 10(b) Moment at spudcan base level

10(c) Horizontal force at spudcan base level

Figure 10. Results of tests on Box 4: fixed leg ($\lambda = 0.5$, $\kappa = 2$, $\eta = 0.33$, $\beta = 18.5^\circ$; Table 1)

666

667 11(a) Non-dimensional moment

668

11(b) Normalised sliding

Figure 11. Effect of perforation: free sliding leg ($\kappa = 2$, $\eta = 0.33$, $\beta = 18.5^\circ$; Box 1~3, Table 1)

677

678 12(a) Non-dimensional moment

679

12(b) Non-dimensional horizontal force

Figure 12. Effect of perforation: fixed leg ($\kappa = 2$, $\eta = 0.33$, $\beta = 18.5^\circ$; Box 4~5, Table 1)