Predicting Punch-through Failure of a Spudcan on Sand Overlying Clay

By

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BEng

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In accordance with the University of Western Australia’s regulations regarding Research Higher Degrees, this thesis is presented as a series of journal papers. The contributions of the candidate and co-author(s) for the papers comprising chapters 3, 4, 5, 6 and 7 are hereby set forth.

**Paper 1**

The paper presented in Chapter 3 is first-authored by the candidate and co-authored by Research Associate Sam Stanier, Professor Mark Cassidy and Assistant Professor Dong Wang, and is published as:


The candidate planned and carried out the experimental programme this paper is based on under the supervision of the above co-authors. Dr. Sam Stanier and centrifuge technician Bart Thompson also contributed to the experimental testing. The candidate analysed the experimental results and wrote the paper under the supervision of the three co-authors.

**Paper 2**

The paper presented in Chapter 4 is first-authored by the candidate and co-authored by Assistant Professor Dong Wang, Professor Mark Cassidy and Research Associate Sam Stanier, and is published as:

The candidate conducted the numerical analyses under the guidance of Assistant Professor Dong Wang. The candidate used the Coupled Eulerian-Lagrangian (CEL) approach in the commercial package Abaqus/Explicit to simulate the continuous spudcan penetration process. The softening clay model was incorporated as a subroutine to consider the strength degradation of clay due to the softening effect. The candidate validated the model against benchmark cases from the literature before the simulations of centrifuge tests and parametric studies. The candidate analysed the numerical results and wrote the paper under the supervision of the above co-authors.

*Paper 3*

The paper presented in Chapter 5 is first-authored by the candidate and co-authored by Research Associate Sam Stanier, Assistant Professor Dong Wang, and Professor Mark Cassidy, and is submitted as:


The candidate designed and carried out the experimental programme on which this paper is based. The experiments were completed using the updated Particle Image Velocimetry apparatus that was developed by Dr Sam Stanier. The candidate summarised the experimental results and wrote the paper under the supervision of the above co-authors.
Paper 4

The paper presented in Chapter 6 is first-authored by the candidate and co-authored by Assistant Professor Dong Wang, Research Associate Sam Stanier, and Professor Mark Cassidy, and is submitted as:


Under the supervision of Assistant Professor Dong Wang, a Modified Mohr-Coulomb (MMC) model was incorporated into Abaqus/Explicit to depict the softening behaviour of very dense, dense and medium dense sand. The candidate then carried out the numerical simulations of centrifuge tests in the literature, followed by the parametric study. Under the supervision of the above co-supervisors, the candidate wrote the paper to report the findings.

Paper 5

The paper presented in Chapter 7 is first-authored by the candidate and co-authored by Research Associate Sam Stanier, Assistant Professor Dong Wang and Professor Mark Cassidy, and is planned to submit as:


The candidate used the full profile prediction method developed in previous chapters to retrospectively predict the experimental results in the literature. The performance of the
prediction method is validated by comparison with the other simplified methods and an interpretation of the industry guideline, by reproduction of the existing centrifuge tests. The candidate analysed the data and wrote the paper.

I certify that, except where specific reference is made in the text to the work of others, the contents of this thesis are original and have not been submitted to any other university.

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ABSTRACT

Mobile offshore jack-up drilling rigs are self-elevating mobile units deployed to drill oil and gas fields in water depths up to ~ 120 m. A typical jack-up unit consists of a buoyant platform and three independent retractable truss-work legs each resting on a footing. These footings are commonly called spudcans and are generally circular or polygonal in shape with diameters between 10 to 20 m and a conical underside. Jack-ups are installed by using their own weight and that of seawater placed in their ballast to push the spudcan footings into the seabed. Installation in seabeds of sand overlying clay can be difficult if the applied load exceeds the capacity of the top sand layer: the spudcan may plunge into the underlying clay layer with sudden and large uncontrolled displacement. This may lead to buckling of the leg or even toppling of the unit. Such incident of jack-up units is referred to as punch-through. This has significant impact on the safety of structures and personnel as well as cost. This study aims to enhance the understanding of the spudcan and conical footing penetration behaviour in sand overlying clay through centrifuge tests and numerical analyses, and to develop an analytical model that predicts the full load-penetration response.

An extensive series of tests have been performed in the drum centrifuge at the University of Western Australia (UWA) to provide evidence for generalisation of the full profile prediction method. These tests are composed of 15 full model tests and 11 half model tests using the Particle Image Velocimetry technique. The full model tests provided the evidence to adjust and improve an existing failure-stress-dependent model to predict the peak penetration resistance, $q_{\text{peak}}$, before a spudcan pushes a frustrum of sand through into an underlying clay layer. The effect of embedment depth was incorporated in the modified failure-stress-dependent model developed. Further, the model was extended for use in medium dense and very dense sands by a more
comprehensive recalibration. The half model tests highlighted the effect of footing shape on the peak resistance and resistance in the underlying clay and were used to observe the failure mechanisms at different penetration depths.

The Coupled Eulerian-Lagrangian (CEL) approach, available in the commercial package Abaqus/Explicit, was used to replicate the continuous spudcan penetration process and capture the formation of the sand plug induced by the advancing spudcan. The classic Mohr-Coulomb and Tresca models were modified and then incorporated into Abaqus to consider the softening behaviour of sand and clay, respectively. An analytical method and corresponding bearing capacity factor for estimating the resistance in the underlying clay were presented based on a large number of numerical and experimental cases.

To facilitate practical application of the outcomes of the research, a simplified design method was proposed to predict the full penetration resistance profile in the stratigraphy of sand overlying clay. The formulation of each critical stage was derived from the failure mechanisms established experimentally and numerically. All the input parameters in the simplified method can be obtained from conventional site investigations. The performance of the method developed in this thesis compared with previous methods reported in design guidelines and the literature is demonstrated by retrospective simulation of all published centrifuge tests. With better predictive capabilities, the full profile prediction method introduced in this thesis will be a useful tool for industry seeking an improvement in the prediction of jack-up behaviour under vertical loading on sand overlying clay stratigraphy.
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I wish to thank members of the jack-up team: Dr. Youhu Zhang and Mr Shah Neyamat Ullah for the fruitful discussion. I also would like to thank PhD student Jingbin Zheng for providing the user subroutines of Modified Mohr-Coulomb and Tresca models for the CEL simulations. Thanks too to the administration staff, Monica and Eileen for arranging my temporary stay and dealing with any administrative work, also Monica and Ivan for handling all the bookings and purchases.

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NOTATION

Roman

$a, b$ .......... Fitting parameters
$A$ ................. The widest cross-sectional area of the footing
$c_v$ ................. Consolidation coefficient
$C$ .................. A constant of the modified failure stress dependent model
$d$ ................. Penetration depth of the footing
$d_{50}$ ................. Median particle size of sand
$d_{base}$ ............ Depth from the bottom of the sand plug to the sand-clay interface
$d_c$ ................. Depth factor for bearing capacity factor $N_c$
$d_{peak}$ .............. Footing depth at peak penetration resistance
$d_{post, peak}$ ...... Footing depth of post-peak penetration resistance
$d_{punch}$ .......... Punch-through distance
$d_{punch, measured}$ . Punch-through distance measured from the centrifuge test
$d_{punch, predicted}$ . Punch-through distance predicted from the full profile prediction method
$D$ ................. Diameter of footing
$D_F$ ................ Distribution factor
$e_{\text{max}}$ .......... Maximum void ratio
$e_{\text{min}}$ .......... Minimum void ratio
$E$ .................. Young’s modulus
$E^*, E_0$ ........ Parameters in the (modified) conceptual model to simplify the algebra
$g$ ................. Gravitational force
$G_s$ ................ Specific gravity
$h_f$ ................. Thickness of the footing at the widest section
$H_{\text{eff}}$ .......... The distance between the spudcan level and the sand-clay interface
$H_{\text{fdn}}$ .......... Height of the composite foundation of footing and sand plug
$H_{\text{plug}}$ .......... Sand plug height
$H_s$ ................. Sand thickness
$I_D$ ................. Relative density of sand
$I_P$ ................. Plastic index
$I_R$ ................. Relative dilatancy index
$k$ .................. Gradient of linear increment of undrained shear strength with depth
The coefficient of mobilised earth pressure acting along the vertical shear plane in the sand in the method of Teh (2007)

$K$ Coefficient of passive earth pressure

$K_p$ Punching shear coefficient

$m$ A constant in Bolton’s (1986) empirical relationships

$N_c$ Bearing capacity factor

$N_{c0}$ Bearing capacity factor of clay at base level of a circular foundation

$N_{c,\text{deep}}$ Bearing capacity factor at 1D depth below the sand-clay interface

$N_{T\text{-bar}}$ Resistance factor for T-bar penetrometer

$N_t$ Bearing capacity factor due to self-weight

$p'$ Mean effective stress

$q$ Penetration resistance

$q_0$ Effective overburden pressure at the depth of the foundation

$q_{\text{clay}}$ Penetration resistance in the underlying clay

$q_{\text{nom}}$ Nominal penetration resistance

$q_{\text{peak}}$ Peak penetration resistance

$q_{\text{peak, calculated}}$ Calculated peak penetration resistance

$q_{\text{peak, measured}}$ Experimental peak penetration resistance

$q_{\text{post, peak}}$ Post peak penetration resistance

$Q$ Natural logarithm of the grain crushing strength (in kPa)

$Q_0$ Surface bearing capacity

$Q_c$ Footing bearing capacity in the underlying clay

$Q_{c0}$ The bearing capacity of the underlying clay

$Q_{\text{clay}}$ Bearing capacity of a footing in uniform clay

$Q_{c, \text{peak}}$ The clay vertical bearing capacity subjected to vertical and inclined loadings within an area of radius $R$

$Q_{\text{peak}}$ Peak bearing capacity

$Q_{\text{so}}$ Shear force acting at the vertical failure plane formed in the sand

$Q_{\text{sand}}$ Bearing capacity of a foundation in uniform sand

$Q_{s, \text{peak}}$ The vertical component of shear force developed along a simplified inclined failure surface in the upper sand layer

$Q_v$ Vertical bearing capacity

$R$ Radius

$R^2$ Coefficient of determination
s .............. The slope of the regression line
\( s_c \) ............ Shape factor for bearing capacity factor \( N_c \)
\( s_u \) ............ Undrained shear strength of clay
\( s_{u0} \) ............ Undrained shear strength of clay at lowest level of the spudcan widest cross-sectional area
\( s_{ua} \) ............ Average value of the shear strength from \( d-h_l \) to \( d+H_{plug} \)
\( s_{ub} \) ............ Undrained shear strength of clay at the base of the composite foundation
\( s_{ui} \) ............ Intact undrained shear strength of clay
\( s_{um} \) ............ Undrained shear strength of clay at mudline or at sand-clay interface
\( s_y \) ............ Shape factor for bearing capacity factor \( N_y \)
\( S_t \) ............ Sensitivity of clay
\( v \) ............ Penetration velocity
\( V \) ............ Normalised penetration velocity
\( V_f \) ............ Volume of the footing
\( W_0 \) ............ Weight of the soil column trapped between spudcan base and the sand-clay interface
\( W_{peak} \) ............ Weight of the sand wedge trapped between the spudcan level and sand-clay interface

**Greek**

\( \alpha \) ............ Roughness factor
\( \alpha_p \) ............ Projection angle
\( \alpha_{side} \) ............ Side adhesion factor
\( \gamma'_c \) ............ Effective unit weight of clay
\( \gamma'_s \) ............ Effective unit weight of sand
\( \delta \) ............ Interface friction angle between the sand and spudcan
\( \delta_{rem} \) ............ Ratio of fully remoulded and initial shear strengths
\( \theta \) ............ Footing conical angle
\( \theta^* \) ............ Skew angle
\( \kappa \) ............ Dimensionless strength increasing parameter for non-homogeneous cohesive soils
\( \mu \) ............ Mean value
\( \xi \) ................. Accumulated absolute plastic shear strain
\( \Delta \xi \) ................. Incremental plastic shear strain
\( \xi_{95} \) ................. Value of \( \xi \) required for the soil to undergo 95\% remoulding
\( \xi_{cv} \) ................. Plastic shear strains corresponding to critical state
\( \xi_p \) ................. Plastic shear strains corresponding to peak friction angle
\( \sigma \) ................. Standard deviation
\( \sigma' \) ................. Vertical effective stress
\( \overline{\sigma'} \) ................. Mean vertical effective stress
\( \phi' \) ................. Friction angle of sand
\( \phi' \) ................. Reduced friction angle due to non-associative flow rule
\( \phi_1 \) ................. Mobilised sand friction angle
\( \phi_2 \) ................. Reduced operative friction angle
\( \phi_{cv} \) ................. Critical state friction angle of sand
\( \phi_{ini} \) ................. Initial value of friction angle
\( \phi_p \) ................. Peak friction angle
\( \psi \) ................. Dilation angle of sand
\( \psi_p \) ................. Peak dilation angle
\( \omega \) ................. Geometric parameter introduced to account for the change in failure mechanism as the spudcan penetration from \( d = 0 \) to \( d = d_{\text{peak}} \).

**Superscripts**

' ................. *Effective stress quantity*

**Subscripts**

c ................. Clay
cv ................. Constant volume
D ................. Density
fdn ................. Composite foundation of footing and sand plug
F ...............Factor
nom...........Nominal
plug...........Sand plug
punch........Punch-through
rem...........Remoulded
R .............Relative
s .............Sand
T-bar.........Value related to or measured using T-bar penetrometer
v...............Vertical
ABBREVIATIONS

<table>
<thead>
<tr>
<th>Abbreviation</th>
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<tr>
<td>AVE</td>
<td>Average</td>
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<tr>
<td>C</td>
<td>Conical footing</td>
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<tr>
<td>CEL</td>
<td>Coupled Eulerian-Lagrangian</td>
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<td>COFS</td>
<td>Centre for Offshore Foundation Systems</td>
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<tr>
<td>COV</td>
<td>Coefficient of variation</td>
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<td>ISO</td>
<td>The International Organisation for Standardization</td>
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<tr>
<td>LL</td>
<td>Liquid limit</td>
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<tr>
<td>LRP</td>
<td>Load reference point</td>
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<tr>
<td>Max</td>
<td>Maximum</td>
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<tr>
<td>MC</td>
<td>Mohr-Coulomb</td>
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<tr>
<td>Min</td>
<td>Minimum</td>
</tr>
<tr>
<td>MMC</td>
<td>Modified Mohr-Coulomb</td>
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<tr>
<td>NC</td>
<td>Normally-consolidated</td>
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<td>NUS</td>
<td>National University of Singapore</td>
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<tr>
<td>OCR</td>
<td>Over-consolidation ratio</td>
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<td>PIV</td>
<td>Particle image velocimetry</td>
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<td>PL</td>
<td>Plastic limit</td>
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<tr>
<td>RITSS</td>
<td>Remeshing and interpolation technique with small strain</td>
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<tr>
<td>S</td>
<td>Spudcan foundation</td>
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<tr>
<td>SNAME</td>
<td>The Society of Naval Architects and Marine Engineers</td>
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<tr>
<td>UWA</td>
<td>The University of Western Australia</td>
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CHAPTER 1. INTRODUCTION

1.1 MOBILE JACK-UP RIGS AND PUNCH-THROUGH FAILURE

Due to the increasing demand of oil and gas worldwide, the offshore industry has been booming in the last two decades. In offshore oil and gas fields, the majority of drillings in water depth up to 120 m are performed from self-elevating mobile jack-up rigs due to their flexibility and cost-effectiveness (Dean 2010). The total number of jack-up drilling rigs in operation (or under construction) was over 310 by the year 2010 and the cost of the total loss of an average jack-up is estimated as $50 million (Jack et al. 2013).

Jack-up rigs are self-elevating mobile platforms with a buoyant hull and three movable and independent legs that can be jacked up and down. A schematic of a jack-up rig is shown in Figure 1.1. Each of the legs is connected to a large footing called a spudcan, which serves as the foundation support to the rig. The spudcan foundations are of

Figure 1.1. Photograph of a typical three-legged jack-up unit (source: www.oilspillsolutions.org)
inverted cone shape and usually polygonal or circular in plane. The conical underside often has a spigot to facilitate initial positioning and provide additional resistance against sliding (see Figure 1.2 for examples of spudcan shapes). The modern spudcan has an equivalent diameter typically between 10 to 20 m, while the bearing pressure on a spudcan is usually around 200–600 kPa (Osborne et al. 2008; Menzies and Roper 2008).

Figure 1.2. Some examples of typical spudcan shapes (re-plotted after Menzies and Roper 2008)

The approximately triangular buoyant hull enables the unit and all attached equipment to be transported to the desired location. On arrival at the drilling site, the legs are lowered through the hull using the jacking system until the spudcan footings rest on the seabed. Under the imposed weight, the hull is jacked up, and the spudcans penetrate into the soil until they bear the self-weight of the rig. Sea water is then pumped into the hull’s ballast tanks as a surcharge for preloading and used to drive the legs deeply into the seabed soil so they will not penetrate further while operations are carried out. This is to ensure an acceptable margin of safety against the design environmental loading is obtained (SNAME, 2008). Since the installation procedure can only take place in calm
water, the load on the spudcan footing is predominantly vertical during the preload stage. After preloading, the additional load is removed by dumping the water and the hull is jacked clear of the water using the jacking system leaving an air gap around 10 to 20 m. This is to ensure that current, tidal and wave loading acts only on the slender legs instead of the hull during exploratory drilling or other operations.

![Penetration resistance vs. penetration depth graph](image)

**Figure 1.3. Illustration of punch-through failure on sand overlying clay**

Spudcan installation in sand overlying clay sites is challenging from a geotechnical perspective due to a limited understanding of the installation of the large spudcan footings into the seabed. A site consisting of sand overlying clay needs extreme attention as a large uncontrolled penetration of the legs may occur due to a sudden reduction in penetration resistance when the spudcan advances through the sand layer into the underlying clay layer. Damage to the jack-up structure may occur during such a large displacement of the leg and in extreme cases it may even cause the jack-up to collapse. This type of event is termed a punch-through failure. Figure 1.3 illustrates a punch-through event during preloading for a spudcan foundation on sand overlying clay.
1.2 THE NEED FOR FURTHER RESEARCH

As the offshore oil and gas industry continues to flourish, ensuring economic and safe use of mobile jack-up units is of great importance. The soil stratigraphy of sand overlying clay has been encountered in many active regions of jack-up operations and punch-through failures continue to be reported. Though there is an increasing concern about punch-through failures, it continues to be a major problem during installation of jack-up platforms (Jack et al. 2013). According to the statistics of the accidents in Figure 1.4, punch-through accidents account for 53% of all the accidents associated with jack-up rigs. The loss or down-time of the rig has significant financial impacts, as the cost for industry are normally between US$1 million to US$10 million per incident (Osborne and Paisley 2002). There is a need for simple and accurate methods to evaluate if there is a risk of a punch-through failure before the spudcan and jack-up are installed. It is also important not to be overconservative with respect to the estimation of peak penetration resistance, which can otherwise cause wasted time by unnecessarily preloading the unit cautiously or expensive seabed or spudcan modifications.

![Diagram](image-url)

**Figure 1.4.** Case histories classified according to the type of accident (after HSE (2004))
1.3 RESEARCH AIMS

The problem of punch-through failure is relatively complex as the response is dependent on the footing geometry and material properties of both sand and clay. In order to enable a safer assessment of spudcans bearing on sand overlying clay soils, a new prediction method is required to improve the prediction of both the potential and severity of punch-through failure before a jack-up rig is taken to site for installation.

This research was therefore undertaken to investigate spudcan penetration behaviour in sand overlying clay, with the aim of establishing a rational framework to predict the potential and severity of punch-through failure. Particular attention is given to predict the peak penetration resistance in the sand layer (at the point of punch-through failure) and then the resistance in the underlying clay layer (when the load is recovered on the footing). The aims of the research covered in this thesis are:

1. To model experimentally the penetration resistance of spudcan penetration in medium dense sand overlying clay to investigate the potential for punch-through failure.

2. To modify and recalibrate a failure-stress-dependent model for predicting peak penetration resistance for both medium dense and very dense sand.

3. To investigate the effect of footing shape on both the peak resistance in the sand layer and the subsequent resistance in the underlying clay layer, and to measure and understand the soil failure mechanisms corresponding to these conditions.

4. To propose a simplified method for the prediction of the full spudcan penetration-resistance profile.
5. To evaluate the effectiveness of the full profile prediction method by comparison with current recommended practices and previous prediction methods using centrifuge testing data reported in the literature.

1.4 RESEARCH STRATEGY

As it is difficult to obtain complete in-situ records of punch-through events, centrifuge tests are a convenient alternative to produce load-penetration profiles while maintaining similar stress levels and thus comparable soil response, especially in the upper sand layer. In this study, an extensive series of centrifuge tests were conducted to investigate the spudcan punch-through problem and to provide relevant experimental data for the new prediction method. The first part of the study investigated the spudcan behaviour in medium dense sand using full spudcan models in the drum centrifuge at the University of Western Australia. Half-footing model tests with particle image velocimetry (PIV) analyses were then conducted to evaluate the soil flow mechanisms during spudcan penetration on sand overlying clay. Accompanied with the centrifuge tests, numerical analyses were performed using the Coupled Eulerian-Lagrangian (CEL) approach to reproduce the whole spudcan penetration process and to validate the combined effect of a range of different factors parametrically. A Modified Mohr-Coulomb (MMC) model and a modified Tresca model were incorporated into the CEL approach to consider the softening behaviours of sand and clay, respectively. The centrifuge tests and numerical analyses together provide sufficient evidence for the proposed full profile prediction method.

1.5 THESIS OUTLINE

In order to achieve the research aims, the thesis is structured as follows.
Chapter 2 is a literature review, with detailed discussion on the recent research of the spudcan punch-through problem. The review details the physical and numerical modelling techniques that have underpinned the new analytical design methods proposed thereafter. The methods for predicting peak penetration resistance recommended in the current industry guidelines SNAME (2008) and ISO (2012) are presented. The full profile prediction methods of Teh (2007) and Lee (2009) are detailed as well.

Chapter 3 presents the experimental programme conducted to investigate the punch-through potential for spudcans in medium dense sands. Details of the experimental equipment, sample preparation and testing methodology are provided, before the experimental results are presented and interpreted. A modified failure-stress-dependent model is proposed and calibrated against the experiments. The model is also compared with the methods recommended in industry guidelines.

Chapter 4 details the CEL approach taken in Abaqus for numerically analysing the continuous spudcan penetration process. The sand is modelled using the Mohr-Coulomb model, while the clay is modelled using a modified Tresca model to account for strain softening. The bearing capacity in the underlying clay layer is well predicted using an expression for the bearing capacity factor of the spudcan and underlying sand plug. A full penetration resistance profile prediction method is proposed based on this expression combined with an existing modified failure stress-dependent model for predicting peak resistance.

Chapter 5 reports the details of half-spudcan visualisation tests investigating the effects of footing shapes on the peak resistance in sand layer and the resistance in clay layer. The soil failure mechanism for different footing shapes during footing penetration is
presented. The analytical equations for the full profile prediction are validated based on the findings.

Chapter 6 discusses how a MMC model can be incorporated as a user subroutine in Abaqus to describe the softening behaviour of dense and medium dense sand. The method is shown to retrospectively simulate the experimental results well. These simulations, as well as an extensive parametric study of cases not considered in the centrifuge testing, provide the evidence to finalise an expression for spudcan bearing capacity factor in the underlying clay. A statistical method is proposed to quantify a range of punch-through distance and provide a measure of severity of punch-through failure.

Chapter 7 presents a comparative study of the full profile prediction method developed in the previous chapters by comparison with the methods recommended in the current industry guideline ISO (2012) and prediction methods available in recent literature. The predictive capabilities of each analytical method are examined and described.

Chapter 8 summarises the main conclusions from this research and provides recommendations for future study.

1.6 REFERENCES


CHAPTER 2. LITERATURE REVIEW

2.1 INTRODUCTION

This chapter provides a brief review of the advancements in the study on spudcan penetration in sand overlying clay. Recent findings from physical investigations, numerical simulations and analytical studies are reported. The first part of the chapter details centrifuge testing on flat footings and spudcans, and the main findings of each study are presented. The second part of this chapter presents numerical simulations of spudcan installation in sand overlying clay utilising three different finite element (FE) methods. Finally, the analytical methods recommended in the current industry guidelines and in the literature are presented and reviewed, highlighting the need to re-appraise the existing methods or propose new analytical methods.

2.2 PHYSICAL MODELLING OF SPUDCAN BEHAVIOUR

2.2.1 Centrifuge testing of Craig and Chua (1990)

Three centrifuge tests with very dense sand ($I_D = 89\%$) and one test with loose sand ($I_D = 24\%$) overlying uniform clay (shear strength $s_u$ ranges from 30 kPa to 45kPa) were conducted in Craig and Chua (1990) to observe the spudcan penetration behaviour. The deep failure mechanism when a spudcan penetrates beyond the sand layer into the underlying clay layer was also investigated, but they did not provide any equations to predict the full penetration profile. Preinstalled spaghetti strips were used to visualise soil deformations by dissecting the spent samples following spudcan penetration. A slightly tapered sand plug with a height that is approximately equal to the initial sand thickness $H_s$ was found to left in the clay layer upon dissection and it was discovered that the sand plug caused a lateral distortion in the clay layer, as highlighted in Figure
2.1. Consequently, Craig and Chua (1990) proposed to incorporate the sand plug side friction into the estimation of the spudcan bearing pressure at large penetration depths.

2.2.2 Centrifuge testing of Teh et al. (2008, 2010)

Teh et al. (2008) detailed experimentally the observed failure mode that was induced during the penetration of a spudcan through the sand into the underlying normally consolidated (NC) clay. The failure mechanism of a half-spudcan model was captured using the particle image velocimetry (PIV) technique coupled with close-range photogrammetric correction of camera-induced distortions. The failure mechanisms at different spudcan penetration depths are provided in Figure 2.2. Three components of the peak resistance were proposed: (a) the shear resistance along the inclined plane in the sand; (b) the clay bearing capacity subjected to pure vertical pressure and (c) the clay bearing capacity subjected to combined vertical-horizontal pressure.
Figure 2.2. Spudcan failure mechanisms at different penetration depths for test T2
(a) full spigot penetration; (b) peak resistance $q_{\text{peak}}$; (c) reduced load; (d) second,
smaller peak; (e) penetrating clay layer; (f) final recorded penetration) (after Teh et al. 2008)
The effects of the geometric and strength conditions of the layered soil on the overall failure mechanism of the spudcan foundation were identified. These findings provide useful references for the validation of numerical and analytical methods for the spudcan installation problem.

Teh et al. (2010) conducted a total of 19 centrifuge model tests at the National University of Singapore (NUS) and the University of Western Australia (UWA) to investigate the full penetration resistance profiles of spudcan footings in the sand overlying normally consolidated clay. The centrifuge test results revealed that both the ratio of the bearing resistance between the upper sand layer and the underlying clay layer and the ratio of the upper sand layer thickness to the spudcan diameter \( H_s/D \), affected the development of the spudcan bearing resistance. The first factor indicated the region of major shearing action, while the second factor was deemed to affect the extension of the failure planes in the underlying clay. Larger \( H_s/D \) (either for an increase in \( H_s \) or a reduction in \( D \)) or sand relative density \( I_D \) yielded a larger peak penetration resistance \( q_{\text{peak}} \) and also a greater reduction in resistance after \( q_{\text{peak}} \).

A full profile prediction method was proposed by Teh (2007) focusing on three key characteristic bearing resistances. The performance of the existing design methods was evaluated using the experimental results.

### 2.2.3 Centrifuge testing of Lee et al. (2013a)

Lee et al. (2013a) detailed an extensive series of 25 flat-based foundations and 5 spudcan foundations penetrating very dense sand overlying kaolin clay samples with six different sand thicknesses, covering sand thickness to foundation diameter ratios of 0.21 to 1.12. The \( s_{\text{um}} \) ranges from 16.3 kPa to 19.1 kPa and strength gradient \( k = 2.1 \) kPa/m. The penetration resistance profile consisted of a peak resistance near the sand surface,
an abrupt post-peak reduction and a gradual increase when the foundation penetrated deeper into the underlying clay layer, as shown in Figure 2.3.

Figure 2.3. Typical penetration resistance profile on sand overlying clay from the centrifuge test (after Lee et al. 2013a)

The measured peak penetration resistances were in the range of 220 to 710 kPa, which fits close to the typical operating range of 200 to 600 kPa reported by Osborne et al. (2008), illustrating that the centrifuge appropriately maintained stress similarity between the small scale models and target prototype problem. By incorporating a modified version of Bolton’s stress-dilatancy relationship (Bolton 1986), the back-calculated values of the stress-level dependent friction and the dilation angles in the sand during peak resistance were derived. A larger peak resistance produced lower friction and
dilation angles, which was consistent with the gradual suppression of the dilatancy under increasing confining stress (Bolton 1986).

Below the top sand layer, the penetration resistance in the underlying clay generally increases with depth. The bearing capacity factor in clay $N_c$ at a depth of $0.5D$ and $1D$ below the sand-clay interface was back-calculated from the experimental results, which was higher than the range of 9-14 for buried flat plates or spudcans in non-uniform clays. There was also an increasing trend for the bearing capacity factor with $H/D$ due to the increasing size of the sand plug trapped under the footing, which enlarged the failure zone and mobilised stronger clay underneath the sand plug.

### 2.2.4 Use of previous centrifuge tests in this thesis

For spudcan penetrating sand overlying clay, 62 centrifuge tests were reported in the literature. More specifically, 4 tests were reported by Teh et al. (2008); 19 tests were reported by Teh et al. (2010); 5 tests were reported by Lee (2009) and 30 tests were reported by Lee et al. (2013a). These are used in this thesis for extending a failure-stress-dependent model and calibrating an empirical factor (Chapter 3), verifying the modified sand and clay models and deriving the expression for bearing capacity factor (Chapter 6) and comparing the peak resistance, full penetration resistance profile and punch-through distance (Chapter 7). A summary of the results and comparisons to the new numerical and analytical methods are provided in Tables in Chapter 3, 4, 6 and 7.

### 2.3 NUMERICAL ANALYSES FOR THE SPUDCAN FOUNDATION ON SAND OVERLYING CLAY

Numerical analyses have also been performed to evaluate the spudcan-bearing pressure in sand overlying clay (Yu et al. 2009; Qiu and Henke 2011; Yu et al. 2012; Tho et al. 2012; Qiu and Grabe 2012). These numerical analyses are useful in providing insights
into the failure mechanisms and the effects of each parameter on the punch-through failure. By comparing the results of the parametric studies and by examining the failure surface from the numerical analyses, the failure mechanisms at different penetration depths were obtained. It is difficult to simulate punch-through failure using traditional small strain FE approaches based on Lagrangian algorithms because the soil around the spudcan undergoes significant deformations, translations and rotations during the penetration process. Distortion of the soil mesh can lead to computational non-convergence, which can be avoided by the large deformation FE approaches, such as the Coupled Eulerian-Lagrangian (CEL) and the remeshing and interpolation technique with small strain (RITSS) approaches. Neither approach makes any prior assumptions about the failure mechanisms, and both can be used to replicate the continuous penetration process of the spudcan on the sand overlying clay. In contrast to the implicit-based RITSS approach, the CEL approach takes advantage of the explicit integration scheme to tackle non-associated soil models.

### 2.3.1 Small strain finite element analysis

The commercially available FE software PLAXIS was used by Lee (2009) and Lee et al. (2013a) to model a circular flat-based foundation of the prototype geometry that was studied in the centrifuge test. The upper and lower soil layers were modelled as a Mohr-Coulomb (MC) material and a Tresca material, respectively, modelling behaviour was consistent with drained penetration in the upper sand layer and undrained penetration in the underlying clay layer. The incremental displacement and the shear strain shadings during the failure stage were presented for a typical simulation. A block of sand was pushed down into the underlying clay with rupture lines of the sand block at a dispersion angle close to the prescribed dilation angle of sand. As shown in Figure 2.4,
this was consistent with the PIV deformation fields observed during the half-spudcan penetration tests of Teh et al. (2008).

Figure 2.4. Comparing incremental displacement between (a) PLAXIS and (b) PIV results (after Lee (2009))

PLAXIS was also used by Lee (2009) to derive the bearing capacity factor when the spudcan penetrates into the underlying clay layer. For simplicity, it was assumed that complete backfilling of the cavity of the spudcan occurred when the footing reached the underlying clay, and the upper sand layer was assumed to place a uniformly distributed surcharge load on the clay. As it did not affect the penetration resistance from the shear strength of the underlying clay, this uniform surface load was not included in the FE model. The composite footing of the spudcan and sand plug was idealised as a cylinder with straight edges. The cylinder was simulated using a weightless elastic model. Two main factors were taken into account in the FE analyses, namely, the thickness of the sand plug beneath the footing and the clay strength, which was taken to increase linearly with depth. Based on the results of an extensive series of small strain FE analyses, two design equations corresponding to shallow and deep failure mechanisms in the underlying clay were established. However, a good fit between the model proposed and
the experimental measurements could only be attained if a range of sand plug height of 0.6–0.9Hs was assumed. There was insufficient experimental (PIV) or numerical evidence (large deformation FE) to confirm the sand plug heights.

### 2.3.2 Analysis with the Remeshing and Interpolation Technique with a Small Strain approach

The RITSS approach was proposed by Hu and Randolph (1998). In the RITSS analysis, the spudcan penetration was divided into a series of small strain Lagrangian steps along with frequent remeshing of the deformed soil. The field variables, including the stresses and material properties, are continuously mapped from the old to the new mesh. Yu et al. (2009) and Hossain and Randolph (2010) implemented the RITSS method to reproduce the spudcan penetration in stiff soil overlying soft clay. The RITSS strategy can be combined with almost all implicit FE codes, but it requires in-house coding for the interpolation of the field variables and mesh regeneration. Yu et al. (2012) used the Mohr-Coulomb (MC) yield criterion to simulate the behaviour of loose sand. The effect of different factors, including the thickness, friction angle and the stiffness of the upper sand and the shear strength of the underlying clay were discussed. The peak penetration resistance was observed to increase with the friction angle of the sand and the sand thickness and to decrease with increasing shear strength of the underlying clay. The post-peak bearing capacity after a deep penetration into the clay layer was found to be linearly proportional to the initial sand-clay shear strength ratio at the sand-clay interface. A simple equation was proposed to correlate the post-peak bearing capacity factor with the soil properties; in this equation, the bearing capacity factor is related to the strength ratio between the sand and the clay.
Li et al. (2013) modified the classical MC model to capture the post-peak of the sand on a critical state basis. The state-dependent friction and dilation angle are considered in their critical state MC model. In a large deformation analysis using RITSS, the critical state MC model offered a prediction that was reasonably well matched to the softening/hardening behaviour of the sand in the laboratory test. Both the peak resistance and the post-peak softening were captured during the large penetration of the spudcan. The dilation angle contours at different spudcan penetration depths are presented in Figure 2.5 to show the evolution of the dilation angle with the stress level (note: valley resistance means the lowest resistance in the clay layer). However, the model was incorporated into a locally developed package AFENA, which is not fully implicit and it was not easy to reach convergence for some large penetration problems.

![Dilatancy angle fields in soils (after Li et al. 2013)](image)

**Figure 2.5. Dilatancy angle fields in soils (after Li et al. 2013)**

### 2.3.3 Review of the Coupled Eulerian-Lagrangian approach

The CEL approach is essentially based on an explicit integration scheme and is available in the commercial package ABAQUS/Explicit. The inertia effect in a real
spudcan penetration is slight due to the slow penetration rates, so a quasi-static calculation is required. The selections of the spudcan penetration velocity and the mesh density need to be tested through parametric studies to ensure that no inertial effects are prevalent in the results. Applications of the CEL approach for continuous spudcan penetration can be found in Tho et al. (2009, 2012 and 2013) and Hu et al. (2012a, b). Tho et al. (2009, 2012) firstly elucidates the mesh density and penetration velocity requirement, and the factors influencing the simulation time. The applicability of the CEL approach was then validated against experimental data for uniform clay with a constant strength, consolidated clay with strength increasing linearly with depth as well as in layered soil profiles involving stiff clay/sand overlying soft clay in Tho et al. (2012). The numerical simulations agreed well with the experimental observations both in terms of the penetration resistances and the soil flow mechanisms. Qiu and Henke (2011) and Qiu and Grabe (2012) later incorporated the hypoplastic and Tresca or visco-hypoplastic constitutive models into the CEL approach to simulate the strain-hardening-softening behaviours of the sand and clay. Both loose and dense sand penetration tests were investigated and the velocity fields were compared with those calculated from the PIV tests of Teh et al. (2008). The hypoplastic model showed good potential for improving the prediction of the bearing capacity of a spudcan penetration in dense sand overlying clay compared with the MC model. However, there are 16 parameters in the hypoplastic model, making it difficult to calibrate the required parameters and to maintain computational stability. Additionally, velocity fields are not an appropriate way to present the flow mechanism because the nodal velocities in the CEL approach are not the physical/material velocities (Dassault, 2011).

2.3.4 Approach of this thesis
This thesis utilises the CEL approach to investigate the bearing pressure of spudcan foundations on sand overlying clay. The behaviour of sand and clay are described with the Modified Mohr-Coulomb and Tresca models, with modifications accounting for the effects of strain softening response of the soil introduced. The CEL results are used to compare with centrifuge tests results and verify the performance of the soil models. The good agreement between centrifuge tests and numerical analyses allows the study to be extended parametrically to cover a wide range of sand relative densities, footing shapes and layer geometries. All the parametric studies consider the cases not covered in the centrifuge tests and encompass most cases of practical interest. These together contribute to the formulation of the penetration resistance in the underlying clay layer and finally the full penetration resistance profile prediction method.

2.4 PREDICTION OF THE PEAK PENETRATION RESISTANCE

Before a jack-up operates at a given site, an assessment of its ability to be installed and preloaded safely must be performed. The bearing capacity of sand overlying clay is usually predicted using simplified analytical models by calculating the force equilibrium of the imaginary sand block that is being pushed into the underlying clay. SNAME (2008) and ISO (2012) provide the current guidelines for the jack-up industry to predict the peak penetration resistance of a spudcan on sand overlying clay soils. Two widely used methods, the load spread method and the punching shear method, are recommended as the basis for the calculation of the peak penetration resistance in the guidelines. This section reviews the methods for the prediction of the peak penetration resistance.

2.4.1 Load spread method
The load spread method (also known as the projected area method) is most commonly used to calculate the foundation bearing pressure in sand overlying clay. The method is based on the assumption that the footing load is spread through the upper sand layer with a projection angle $\alpha_p$ to the surface of the underlying clay over a larger imaginary area, as shown in Figure 2.6(a). The peak penetration resistance is determined from the bearing capacity of the underlying clay with enlarged imaginary area at the sand-clay interface (i.e., the resistance generated by shearing of the sand is ignored). For full or partial back-flow, the ultimate vertical capacity of a circular footing can be computed using:

$$q_{\text{peak}} = \left(1 + 2 \frac{H}{D} \tan \alpha_p \right)^2 \left(s_u s_u s_u d_c + q_0 - H_s \gamma_s'\right)$$  \hspace{1cm} (2.1)

where $s_c$ is the shape factor and $d_c$ is the depth factor, $s_u$ is the undrained shear strength of the underlying clay (or the clay strength at the sand-clay interface $s_{um}$ for the consolidated clay), $q_0$ is the effective surcharge, and $\gamma_s'$ is the effective unit weight of sand. The value of $\tan \alpha_p$ is crucial in this method. For a conservative design, values of 1/5 and 1/3 were recommended by SNAME (2008). The load spread method has oversimplified the actual mechanism by ignoring the frictional resistance in the sand layer. In addition, the effects of various footing geometries are not accounted for, and only a predefined projection angle is required. This method tends to under-predict the bearing capacity and therefore overstates the potential for punch-through to occur with respect to the planned preload value.
2.4.2 Punching shear method

As an alternative, ISO (2012) recommends the punching shear method to calculate \( q_{\text{peak}} \).

The punching shear method is based on Meyerhof (1974) and Hanna and Meyerhof (1980) and has a failure mechanism comprising a vertically sided block beneath the circular footing that punches into the underlying clay. Unlike the load spread method, the frictional resistance of the slip surface is taken into account in the calculation of the bearing capacity for sand overlying clay soils. For simplicity, a cylindrical shear surface is used. Considering the force equilibrium shown in Figure 2.6(b), the peak resistance for a circular footing is:

\[
q_{\text{peak}} = (s_u N_c s_d + q_0) - H_s \gamma'_s + 2 \frac{H_s}{D}(H_s \gamma'_s + 2q_0) K_s \tan \phi'
\]  

(2.2)

where \( \phi' \) is the friction angle of the sand. The punching shear coefficient \( K_s \) is introduced to compute the frictional resistance on the assumed vertical slip planes. \( K_s \) depends on the friction angle \( \phi' \) and the bearing capacity ratio \( Q_{\text{clay}}/Q_{\text{sand}} \), where \( Q_{\text{clay}} \) and \( Q_{\text{sand}} \) are the bearing capacity of a foundation in uniform clay and uniform sand,

![Figure 2.6. Load spread method and punching shear method for circular footings](image)
respectively. $K_s$ can be determined from the design chart in Figure 2.7. The formulae used to calculate $Q_{\text{sand}}$ and $Q_{\text{clay}}$ for the circular footing are:

$$
Q_{\text{clay}} = s_u N_s s_c d_c A; \quad Q_{\text{sand}} = s_u N_s \frac{\gamma'}{2} d_A
$$

(2.3)

where $s_u$ is the ultimate strength of the soil, $s_c$ is the cohesion, $s_u$ is the unit weight of the soil, $N_s$ is the bearing capacity factor for the self-weight, and $A$ is the cross-sectional area of the circular footing. Due to the limited range of the design chart for parameter $K_s$, interpolation or extrapolation is necessary for the cases that are beyond the given values in Figure 2.7. As an alternative, a lower bound of $K_s \tan \phi' = 3s_u/D\gamma'$, is recommended in SNAME (2008), resulting in an expression for $q_{\text{peak}}$ that is independent of the friction angle $\phi'$.

![Figure 2.7. Relationship between the bearing capacity ratio and the punching shear coefficient (replotted after ISO (2012))](image)

2.4.3 Other prediction methods

In addition to the above analytical methods, Shiau et al. (2003) used finite element formulations of limit analysis to obtain both the upper and lower bound solutions for the
bearing capacity in sand overlying clay. Upper bound plasticity analyses were also used
by Florkiewicz (1989) and Michalowski and Shi (1995) to calculate the bearing pressure
of strip foundations on layered soils. Since they were all based on plane strain analyses,
they are unsuitable for application to the axisymmetric spudcan problem.
Okamura et al. (1998) proposed a limit equilibrium method to evaluate the footing
bearing pressure in sand overlying clay. It combined the theories of the load spread
method and the punching shear method. The failure mechanism mainly consists of a
general shear failure across the enlarged area at the surface of the underlying clay and
the forces that act on the sides of a sand block formed between the footing base and the
sand-clay interface. However, this method was developed based on centrifuge tests with
small prototype foundation diameters (0.8-3 m) and with relative thin sand layers. In
addition, it does not totally account for the stress-level dependency of friction angle
because the effects from the footing size and the undrained shear strength of the
underlying clay are not incorporated. Both Teh et al. (2010) and Lee et al. (2013b) have
demonstrated that this method tends to over-estimate $q_{\text{peak}}$ values. Therefore, this
method is not used for comparison with the other prediction methods.

2.5 FULL PENETRATION RESISTANCE PROFILE PREDICTION

Teh (2007), Lee (2009) and Lee et al. (2013a, b) proposed full penetration resistance
profile (from spudcan tip touchdown to the final penetration depth) prediction methods
in sand overlying clay based on observations from centrifuge tests and numerical
analyses, respectively. This section details the above analytical methods for the
comparison of their performance in the following chapters.

2.5.1 The Teh method
Based on the limit equilibrium theory, Teh (2007) proposed a method to generalise the soil bearing capacity–depth profile using three different phases, namely, the surface bearing capacity ($Q_0$ when the widest cross-sectional area of the spudcan is first in contact with the sand, i.e., penetration depth $d = 0$), the peak bearing capacity ($Q_{\text{peak}}$ at $d = d_{\text{peak}}$, where $d_{\text{peak}}$ is the depth of the peak penetration resistance), and the bearing capacity in the underlying clay layer ($d \geq H_s$).

The failure mechanisms established for each phase are restricted to the case of the two-layered failure mode involving the mobilisation of the strength in both the sand and clay layers, as shown in Figure 2.8. The bearing capacities in the first two phases depend on the strength variance between the upper sand layer and the lower clay layer and the ratio of the sand layer thickness to the spudcan diameter. When the spudcan penetrates into the underlying clay layer, the bearing capacity is significantly affected by the sand plug that is trapped underneath the penetrating spudcan.

Figure 2.8(a) illustrates the failure mechanism for $Q_0$, and the formulations to calculate $Q_0$ are as follows:

$$Q_0 = Q_{s0} + Q_{c0} - W_0$$  \hspace{1cm} (2.4)

$$Q_{s0} = \frac{\pi D^2}{2} \gamma_s' H_s \sin \phi$$  \hspace{1cm} (2.5)

$$Q_{c0} = \pi \left( \frac{D}{2} \right)^2 \left[ N_c \gamma_{\text{um}} + \gamma_s' H_s \right]$$  \hspace{1cm} (2.6)

$$W_0 = \pi \left( \frac{D}{2} \right)^2 \gamma'_s H_s$$  \hspace{1cm} (2.7)

where $Q_{s0}$ is the shear force that acts at the vertical failure plane that is formed in the sand; $Q_{c0}$ is the bearing capacity of the underlying clay; $W_0$ is the weight of the soil column trapped between spudcan base and the sand-clay interface; and $K$ is the
coefficient of the mobilised earth pressure acting along the vertical shear plane in the sand. The mobilised sand friction angle \( \phi_h \) is obtained by iterating between the mean in-situ effective stress, \( p' \), in the sand layer and the stress-dilatancy correlation proposed by Bolton (1986). \( \phi_h \) varies with \( H_s, \gamma'_s \), the relative density of sand \( I_D \) and the sand critical state friction angle \( \phi_{cv} \).

\[
\phi_h = \phi_{cv} + 3I_R = \phi_{cv} + 3\left[I_D \left(Q - \ln p'\right) - 1\right] \tag{2.8}
\]

\[
p' = \frac{1}{4} \gamma'_s H_s \left(1 + K_p\right) \tag{2.9}
\]

\[
K_p = \frac{1 + \sin \phi}{1 - \sin \phi} \tag{2.10}
\]

where \( \phi_{cv} \) is the critical state friction angle of sand; \( I_R \) is the relative dilatancy index; \( I_D \) is the relative density of sand; \( Q \) is the natural logarithm of the grain crushing strength (in kPa) and \( K_p \) is the coefficient of passive earth pressure. Given that the underlying clay is not directly acted upon by the spudcan but through the base of the upper sand layer, \( N_c \), for a rough flat footing was used as proposed by Houlsby and Martin (2003).

The maximum bearing load \( Q_{peak} \) is calculated using the equations:

\[
Q_{peak} = Q_{s,peak} + Q_{c,peak} - W_{peak} \tag{2.11}
\]

\[
Q_{s,peak} = \pi \gamma'_s K_p \sin(\phi_h - \omega) \left[ d_{peak} + \frac{1}{2} H_{eff} D H_{eff} \right] \tag{2.12}
\]

\[
Q_{c,peak} = \pi \left( N_{s,um} s + H_s \gamma'_s \right) \left[ R^2 - \frac{0.5}{R - r} \left( \frac{2}{3} R^3 + \frac{1}{3} r^3 - R^2 r \right) \right] \tag{2.13}
\]

\[
W_{peak} = \frac{1}{3} \pi H_{eff} \left( \frac{D}{2} \right)^2 + R \frac{D}{2} + R^2 \gamma'_s \tag{2.14}
\]
Shearing along vertical plane

General bearing shear failure

Sand
Clay

(a) $Q_0$

Shearing along logarithmic spiral failure surface

Clay bearing shear failure subjected to vertical and inclined loads

Sand
Clay

(b) $Q_{\text{peak}}$

Sand plug
Side friction

Deep flow mechanism in clay

(c) $Q_c$

Figure 2.8. Schematic of the failure mechanisms of the Teh method
where $Q_{s,\text{peak}}$ is the vertical component of the shear force developed along a simplified inclined failure surface in the upper sand layer; $Q_{c,\text{peak}}$ is the clay vertical bearing capacity subjected to the vertical and inclined loadings within an area of radius $R$; and $W_{\text{peak}}$ is the weight of the sand wedge trapped between the spudcan level and the sand-clay interface; $\phi_s$ is the reduced operative friction angle taken as an average between $\phi_h$ and $\phi_{cv}$; $H_{\text{eff}}$ is the distance between the spudcan level and the sand-clay interface, i.e., $H_{\text{eff}} = H_s - d_{\text{peak}}$; $\omega$, $r$ and $R$ are the geometric parameters of the failure planes that determine the value of $Q_{s,\text{peak}}$, $Q_{c,\text{peak}}$ and $W_{\text{peak}}$ and account for the change in the failure mechanism as the spudcan penetrates from $d = 0$ to $d = d_{\text{peak}}$. The values of $\omega$, $r$ and $R$ are given in the design charts in Teh (2007).

By taking the spudcan together with the sand plug as a composite foundation in which the design depth is shifted from the spudcan level to the bottom of the sand plug and assuming the composition is flat and rough with a side friction force acting along the periphery of the sand plug, the expression for calculating the bearing capacity of the underlying clay $Q_c$ was proposed as:

$$Q_c = s_{ub}N_c A + \frac{4 s_{ua} \alpha_{\text{side}} (H_{\text{plug}} + h_t) A}{D} + \gamma'_c V_f$$

(2.15)

where $H_{\text{plug}}$ is the height of the trapped sand plug; $s_{ub}$ is the clay strength at the base of the composite foundation; $h_t$ is the thickness of the spudcan at its widest section; $s_{ua}$ is the average value of the shear strength from $d - h_t$ to $d + H_{\text{plug}}$; $\alpha_{\text{side}}$ is the side adhesion factor taken as unity (Craig and Chua, 1990); $\gamma'_c$ is the effective unit weight of the underlying clay; $V_t$ is the volume of the spudcan body, and $A$ is the widest cross-sectional area of the spudcan. The bearing capacity coefficient can be determined using the equation proposed by Hossain et al. (2006) as:
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\[ N_c = 10 \left( 1 + 0.075 \frac{d + H_{\text{plug}}}{D} \right) \quad N_c \leq 11.5 \quad (2.16) \]

By connecting \( Q_0, Q_{\text{peak}} \) and \( Q_c \), a simplified bearing capacity-depth profile is produced.

### 2.5.2 The Lee et al. method

Lee (2009) and Lee et al. (2013a, b) proposed new analytical methods to predict both the peak penetration resistance and the full penetration resistance profile of the circular foundations. A simplified analytical model was developed to predict the peak resistance, assuming that when \( q_{\text{peak}} \) occurs, a sand frustum with a dispersion angle equal to the dilation angle \( \psi \) is pushed into the underlying clay as shown in Figure 2.9(a). The footing pressure and the weight of the sand frustum are resisted by the frictional resistance along the sides of the sand block and the bearing capacity of the underlying clay. The resulting equation for \( q_{\text{peak}} \) is:

\[
q_{\text{peak}} = \left( N_{c0} s_{\text{um}} + q_0 \right) \left( 1 + \frac{2H}{D} \tan \psi \right)^{E^*}
\]

\[
+ \frac{\gamma' D}{2 \tan \psi (E^* + 1)} \left[ 1 - \left( 1 - \frac{2H}{D} E^* \tan \psi \right) \left( 1 + \frac{2H}{D} \tan \psi \right)^{E^*} \right]
\]

where \( N_{c0} \) is the bearing capacity factor of the clay at the foundation base, which is obtained using the relationship proposed by Houlsby and Martin (2003) for circular foundations with shear strength increasing linearly with depth and is expressed as:

\[
N_{c0} = 6.34 + 0.56 \frac{k \left( D + 2H \tan \psi \right)}{s_{\text{um}}}
\]

where \( k \) is the gradient of the linear increment of the undrained shear strength with depth. \( E^* \) is a term adopted to simplify the algebra and is equal to:

\[
E^* = 2 \left[ 1 + D_H \left( \frac{\tan \phi^*}{\tan \psi} - 1 \right) \right]
\]


Figure 2.9. Schematic of the failure mechanisms of the Lee et al. method
where $D_F$ is an empirical distribution factor. Equation 2.15 was derived from the integration of the vertical force equilibrium equation on an infinitesimal disc element within the conceptual sand frustum (see Figure 2.9(a)), in which $D_F$ was intended to approximate the relationship between the local stress along the failure surface and the average vertical effective stress. The distribution factor $D_F$ is the only empirical parameter in these equations to be determined from back-calculation of testing results. Based on the centrifuge test results of 25 flat circular foundation and 5 spudcan penetration tests on sand overlying clay, it was found that the back-calculated values of $D_F$ vary linearly with the $H_s/D$ ratio, with the empirical relationships of:

$$D_F = 0.726 - 0.219 \frac{H_s}{D} \begin{cases} \text{for flat footings and:} \\ \frac{H_s}{D} \leq 1.12 \end{cases}$$

(2.20)

for spudcan (13° conical underside). It can be seen that the spudcan has higher $D_F$ values than the flat footing at similar $H_s/D$. The reason lies in a conical spudcan invokes higher lateral stress than a flat footing does, so higher shear stress along the slip surface. This is in turn reflected in higher $D_F$ values for spudcan. $\phi'$ is the reduced friction angle due to the non-associated flow rule and is calculated as:

$$\tan \phi' = \frac{\sin \phi' \cos \psi}{1 - \sin \phi' \sin \psi}$$

(2.22)

When penetration continues after reaching $q_{\text{peak}}$, the resistance reduces abruptly to a minimum value termed the post-peak penetration resistance $q_{\text{post, peak}}$, at which the sand is shearing at a critical state ($\phi' = \phi_{cv}$). The corresponding failure mechanism is
illustrated in Figure 2.9(b). The equation for calculating the value of $q_{\text{post, peak}}$ can be expressed after simplification as:

$$q_{\text{post, peak}} = \left[ d_c N_c \sigma_{\text{um}} + \left( \gamma_c' + \gamma_s' \right) d_{\text{post, peak}} \right] e^{E_0}$$

where $d_c$ is the depth factor for the bearing capacity factor $N_c$, which is calculated as:

$$d_c = 1 + 0.4 \frac{d_{\text{post, peak}}}{D}$$

Parameter $E_0$ is also adopted to simplify the algebra and expressed as:

$$E_0 = 4D \sin \phi'_{cv} \frac{(H_s - d_{\text{post, peak}})}{D}$$

where $d_{\text{post, peak}}$ is the depth of the post-peak penetration resistance. The measurements of the 25 flat footing tests indicate the values of $d_{\text{post, peak}}$ fall in the range of 20% to 40% of the sand thickness. For design purposes, it is adopted as:

$$d_{\text{post, peak}} = 0.3H_s$$

For the bearing capacity factor in the underlying clay, two expressions for the bearing capacity factor were proposed, corresponding to the shallow and deep mechanisms as follows:

$$N_c \text{ for the shallow mechanism: } N_c = 4 \frac{d_{\text{base}}}{D} + 9 \text{ as } \frac{d_{\text{base}}}{D} \geq \frac{H_{\text{fdn}}}{D}$$

where $d_{\text{base}}$ is the depth from the sand-clay interface to the base of the composite foundation, and $H_{\text{fdn}}$ is the height of the composite foundation for the footing and the trapped sand. Both are shown in Figure 2.9(c).
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\[ N_c \text{ for the deep mechanism:} \]
\[ N_c = \left[ 1 - \frac{1}{2} \kappa \frac{H_{\text{fdn}}}{D} \right] \left( 18.2 \sqrt{\frac{H_{\text{fdn}}}{D}} + 0.7 - 2 \right) \quad (2.28) \]
\[ as \quad \frac{H_{\text{fdn}}}{D} \leq 1.12 \quad and \quad \frac{d_{\text{base}}}{D} > \frac{H_{\text{fdn}}}{D} + 0.5 \]

where \( \kappa (= kD/s_{\text{un}}) \) is the dimensionless strength increasing parameter for the non-homogeneous cohesive soils.

\[ N_c = \text{minimum} (N_c \text{ for shallow mechanism, } N_c \text{ for deep mechanism}) \quad (2.29) \]

The penetration resistance in the underlying clay \( q_{\text{clay}} \) can then be calculated using the following expression:

\[ q_{\text{clay}} = N_c s_{\text{ub}} + \gamma'_c H_{\text{fdn}} \quad (2.30) \]

A penetration resistance profile in the underlying clay can therefore be constructed using a series of increasing values of \( d_{\text{base}}/D \) as the footing penetrates the clay. A complete simplified penetration resistance profile can be constructed by linking the critical points of \( q_{\text{peak}}, q_{\text{post, peak}} \) and \( q_{\text{clay}} \) with straight lines.

2.6 CONCLUDING REMARKS

Sections 2.2 to 2.5 review the issues related to the bearing capacity of sand overlying clay. Experimental and numerical studies have been carried out to investigate the bearing capacity of sand overlying clay soils, and advanced analytical methods have been developed to predict the full penetration resistance profile. In sections 2.4 and 2.5, a brief review of the different analytical methods for calculating the bearing capacity on the sand overlying clay has been presented. Initial solutions were based on simple methods to calculate the peak penetration resistance, namely, the load spread method and the punching shear method. Both methods are still commonly used in industry (through SNAME (2008) and ISO (2012)), though the predictions are often found to be
unsatisfactory in predicting $q_{\text{peak}}$ (when compared to centrifuge data) (Teh et al. 2010; Lee et al. 2013b). In addition, they are not intended for a successive assessment of the spudcan bearing pressure in sand overlying clay undergoing continuous penetration, and in many cases the mechanisms assumed do not resemble the true failure mechanisms induced by spudcan penetration observed using PIV (Teh et al. 2008). For the calculation of the peak penetration resistance, both the Teh method and the Lee et al. method were based on limit equilibrium theory and incorporated different assumptions and simplifications. Though the methods of Teh and Lee et al. are based on observed soil failure mechanisms and were devised to evaluate the spudcan bearing resistance in sand overlying clay, these approaches were mainly derived from tests on very dense sands. In addition, the bearing capacity factors used in both methods were either adopted from a single clay layer case or derived from small strain finite element analyses. Their performance for medium and loose sand and for various footing shapes requires further investigation.

Attention should also be paid to the aspects of real soil behaviour, for example: layered soil profiles, stress dependent strength in particular. A full profile prediction method that balances accuracy and practicality is appealing to the offshore geotechnical engineering community. Furthermore, such a prediction method should only rely on parameters that can be derived from commonly performed offshore site investigation tests. The following chapters detail the gradual development of a full penetration resistance profile prediction method that meets these two criteria.

### 2.7 REFERENCES


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CHAPTER 3. PREDICTING PEAK RESISTANCE OF SPUDCAN PENETRATING SAND OVERLYING CLAY THROUGH CENTRIFUGE MODEL TESTING

ABSTRACT

Accurately predicting peak penetration resistance $q_{\text{peak}}$ during spudcan installation into sand overlying clay is crucial to an offshore mobile jack-up industry still suffering regular punch-through failures. This paper describes a series of spudcan penetration tests performed on medium dense sand overlying clay and compares the response to existing centrifuge data from tests performed on very dense sand overlying clay. Together these data demonstrate that punch-through is a potential problem for both very dense and medium dense sand overlying clay soil stratigraphies. Using this experimental database, a failure-stress-dependent model has been modified to account for the embedment depth, and the depth of occurrence of $q_{\text{peak}}$ is shown to be a function of the sand thickness $H_s$. The model then was recalibrated, taking these findings into account, for a larger range of material properties and ratios of sand thickness to spudcan diameter $(H_s/D)$. Finally, the performances of the modified and recalibrated model and the methods given in the current guidelines are verified by comparing their predictions against centrifuge model tests data. The comparisons show that the modified model yields more accurate predictions of $q_{\text{peak}}$ over the range of $H_s/D$ ratios of practical interest, which when used in practice will potentially mitigate the risk of unexpected punch-through on sand overlying clay stratigraphies.
3.1 INTRODUCTION

Modern jack-up structures typically consist of a triangular platform with three legs that are jacked through the deck into the seabed. A jack-up is then installed by filling water ballast tank on the platform, which pushes the legs and large inverted conical spudcan footings attached at the ends into the seabed soil. The preloading procedure continues until the spudcans have been effectively proof tested, at which time the ballast water is dumped, and the platform is jacked up above the water surface for operation. When jack-up platforms are installed on seabed sediments consisting of a sand layer overlying soft clay, there is the potential for punch-through failure, where the spudcan footing pushes the stronger layer into the softer layer. This can cause vertical displacement of one or more of the legs of the platform in a rapid and uncontrolled manner, which as a consequence can lead to buckling of the legs or in extreme cases even toppling of the platform. The cost of these incidents is estimated to be between US$10 million and US$30 million per incident, and such incidents continue to be problematic (Hossain and Safinus 2012). Therefore, accurately predicting the peak penetration resistance and thus the potential for punch-through failure is an important issue for jack-up platform operators both for operational safety and for field-development economics.

Craig and Chua (1990) performed a series of centrifuge tests investigating the potential for punch-through of foundations on sand overlying clay. They observed that a peak penetration resistance was attained relatively rapidly, which was followed by reducing penetration resistance that caused rapid leg penetration. Cutting of the samples along the central cross section after spudcan extraction exposed a slightly downward-tapering plug of sand with depth approximately equal to the sand-layer height. In contrast, an inverted truncated-cone sand plug was visualized by Teh et al. (2008) using the particle-
image velocimetry technique (White et al. 2003) in a centrifuge, and the failure mechanism of spudcan foundations on sand overlying clay was discussed. Teh et al. (2010) proposed that the bearing-resistance-depth profile of a punch-through event can be determined by three characteristic bearing resistances and corresponding depths. Based on an extensive series of flat-footing and spudcan penetration tests on very dense sand overlying clay and the observations of Teh et al. (2008), Lee et al. (2013b) proposed a failure stress-dependent model to calculate the peak penetration resistance $q_{\text{peak}}$. However, the model was calibrated solely using experimental data for very dense sand overlying clay and for limited geometries of spudcan foundations. Its performance therefore requires further validation. Of great concern is whether recalibration is needed for looser sands overlying clay soil conditions.

The aims of this paper are:

1. To model experimentally the penetration resistance of a spudcan of generalised geometry penetrating medium dense sand overlying clay in the centrifuge and to access the potential for punch-through failure;

2. To extend the stress-dependent model for predicting peak penetration resistance of Lee et al. (2013b) to (a) both medium dense and very dense sands and (b) account for the embedment depth of the peak penetration resistance;

3. To recalibrate the modified model based on the experimental results;

4. To investigate the ability of the modified model to predict peak penetration resistance during spudcan foundation installation on sands overlying clay; and

5. To assess the performance of the modified model by comparison with current recommended practices using centrifuge data reported in the literature.
3.2 EXPERIMENTAL SET UP

Physical modelling of spudcan penetration on medium dense sand overlying clay was conducted using the drum centrifuge at the University of Western Australia (UWA), a detailed description of which was reported by Stewart et al. (1998). Spudcan foundations in practice are typically circular with diameters of 10–20 m. In this investigation, a generalised-geometry spudcan as illustrated in Figure 3.1 was used and was referred to as the UWA spudcan. The spigot angle of 76° and main conical angle of 13° were kept constant for different model spudcan diameters to ensure that any geometric impacts remained consistent between tests. Table 3.1 contains a summary of the prototype spudcan geometries tested in this investigation. This UWA spudcan was identical to the model spudcan in very dense sand overlying clay by Lee et al. (2013a).

![Figure 3.1. Generalised UWA spudcan geometry](image)

Commercially available superfine silica sand and kaolin clay were adopted in all the centrifuge tests to form the sand and clay layers, respectively. Both materials have been well characterised and used extensively in the geotechnical centrifuges at UWA. The key properties of the sand and clay are summarised in Cheong (2002) and Stewart (1992), respectively.
The kaolin clay was mixed into a slurry with a water content of 120%. It was then placed in the drum channel using the actuator at an acceleration of 20g until the channel was full before being normally consolidated at an acceleration of 300g. After consolidation, the normally consolidated clay was scraped back, leaving a nonzero shear strength at the same surface and the desired target clay thickness of 150 mm. A fabric membrane was then placed on top of the clay, and sand was pluviated into the channel under an acceleration of 20g using a specially designed sand placement tool. Medium dense sand was formed by pluviating the fine particles through a layer of water kept on top of the sample. The underlying clay was then lightly overconsolidated at an acceleration of 300g. The sand and fabric membrane were removed before the sand was laid again following the same procedure but without the fabric membrane. The fabric membrane was used to facilitate removal of the surcharging sand layer, which was disturbed during normal consolidation of the underlying clay layer, allowing relaying of a new, undisturbed sand layer for testing. A target sand thickness was achieved by scraping the sand surface down to the desired height by means of a scraping plate attached to the actuator. All tests were conducted at 200g, and the overconsolidation ratio (OCR) for underlying clay was at least 1.5.

### 3.3 TESTING PROCEDURE

Fifteen spudcan penetration tests were performed on medium dense sand overlying clay with sand thickness $H_s$ of 16, 25, and 30 mm. Five tests were conducted at each sand thickness, as detailed in Table 3.1. The ratio of $H_s/D$ was then between 0.16 and 1.0, which covers the range of practical interest because few punch-through failures have been reported for $H_s/D > 1$. The first five tests were performed with a sand thickness of 30 mm. Following these tests, the sand was further scraped back to 25 mm and then to
16 mm (the bottom clay was reconsolidated under 200g overnight after each scraping process), allowing tests to be performed at three sand thicknesses. At each stage, the sand was removed over the entire drum channel, but the tests were conducted in different and untouched sites (with a distance of at least $1.5D$ between each test).

### Table 3.1. Relevant prototype parameters for centrifuge tests in this investigation

<table>
<thead>
<tr>
<th>Test Name</th>
<th>Geometry</th>
<th>Sand</th>
<th>Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$H_s$ (m)</td>
<td>$D$ (m)</td>
<td>$A$ (m$^2$)</td>
</tr>
<tr>
<td>L1SP1</td>
<td>6</td>
<td>6</td>
<td>28.27</td>
</tr>
<tr>
<td>L1SP2</td>
<td>6</td>
<td>8</td>
<td>50.27</td>
</tr>
<tr>
<td>L1SP3</td>
<td>6</td>
<td>10</td>
<td>78.54</td>
</tr>
<tr>
<td>L1SP4</td>
<td>6</td>
<td>12</td>
<td>113.10</td>
</tr>
<tr>
<td>L1SP5</td>
<td>6</td>
<td>14</td>
<td>153.94</td>
</tr>
<tr>
<td>L2SP1</td>
<td>5</td>
<td>6</td>
<td>28.27</td>
</tr>
<tr>
<td>L2SP2</td>
<td>5</td>
<td>10</td>
<td>78.54</td>
</tr>
<tr>
<td>L2SP3</td>
<td>5</td>
<td>14</td>
<td>153.94</td>
</tr>
<tr>
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<td>201.06</td>
</tr>
<tr>
<td>L2SP5</td>
<td>5</td>
<td>20</td>
<td>314.16</td>
</tr>
<tr>
<td>L3SP1</td>
<td>3.2</td>
<td>6</td>
<td>28.27</td>
</tr>
<tr>
<td>L3SP2</td>
<td>3.2</td>
<td>8</td>
<td>50.27</td>
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<tr>
<td>L3SP3</td>
<td>3.2</td>
<td>12</td>
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<tr>
<td>L3SP4</td>
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<td>16</td>
<td>201.06</td>
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<tr>
<td>L3SP5</td>
<td>3.2</td>
<td>20</td>
<td>314.16</td>
</tr>
</tbody>
</table>

* $s_{um}$ is at the sand-clay interface.

The relative density $I_D$ of the sand layer was determined by extracting four samples from equidistant radial locations in the channel using 60-mm-diameter sampling tubes after surcharging and before the tests (at 1g). The samples were collected from the bed carefully, ensuring minimal disturbance, and yielded an average relative density of 43% with a SD of 7%, indicating relatively uniform medium dense sand. The submerged unit weight of the sand $\gamma'_s$ was found to be 10 KN/m$^3$. The submerged unit weight of the clay $\gamma'_c$ was measured directly on 20-mm-diameter samples extracted by a tube sampler and was found to be 7.1 KN/m$^3$ on average.
The spudcans were loaded using displacement control at a constant penetration rate. The penetration rates were determined such that drained behaviour in sand and undrained behaviour in clay were attained. The following normalised penetration rate \( V \) was widely adopted to describe the drainage condition (Finnie and Randolph 1994):

\[
V = \frac{vD}{c_v}
\]  

(3.1)

where \( v \) = penetration velocity of footing; \( D \) = footing diameter; and \( c_v \) = consolidation coefficient. For undrained conditions in clay, there is a transition range of \( 30 < V < 300 \) over which partial drainage is minimised (Finnie and Randolph 1994). The dimensionless velocity \( V \) was maintained as 120 in the clay \([c_v = 2 \text{ m}^2/\text{year} \text{ (Stewart 1992)}]\) for all tests by varying the penetration velocity accordingly. Thus the penetration velocity \( v \) for \( D = 30 \) and 100 mm were 0.254 and 0.076 mm/s, respectively. Silica sand has been estimated to have a \( c_v \) of at least 60000 m\(^2\)/year (Lee et al. 2013a), and therefore, \( V \) was less than 0.01 in the sand layer for the penetration rates and spudcan sizes used. This ensured fully drained behaviour in the sand layer.

To obtain the undrained shear-strength profile of clay layer, T-bar penetrometer tests were performed on the clay layer in isolation following careful removal of the upper sand layer after all the spudcan penetration tests had been completed. This was intended to eliminate the influence of the entrapped sand beneath the penetrometer and avoid potential damage to the penetrometer that may have occurred if it penetrated through the sand layer. An intermediate roughness T-bar factor of 10.5 was assumed. Given that the OCR increased as the sand was scraped away at intervals during the test schedule to allow for thinner sand thickness, the shear-strength profile of clay measured with the
sand removed was calculated using the following relationship to account for the impact of changes in OCR on the shear strength (Koutsotas and Ladd 1985):

\[
\frac{s_u}{\sigma'_v} = a \text{OCR}^b
\]  

(3.2)

where \(s_u\) = undrained shear strength; \(\sigma'_v\) = vertical effective stress; and \(a\) and \(b\) = fitting parameters.

![Figure 3.2. Shear-strength profile from two T-bar penetration tests (\(N_{T\text{-bar}} = 10.5\) demonstrating sample uniformity](image)

Two example T-bar tests from different locations within the drum channel are presented in Figure 3.2, indicating excellent sample uniformity. The OCR profiles for each of the three layer thicknesses were calculated using the measured effective unit weight of the sand and clay layers, prototype dimensions, and the consolidation \(g\) level. The best fit of Equation 3.2 to the T-bar penetrometers profiles for all three sand-layer heights tested was found with values for \(a\) and \(b\) of 0.16 and 0.74, respectively. Thus the sand-clay interface shear strengths and shear-strength gradients for the underlying clay layers
tested were estimated using linear best fits to the nonlinear profiles estimated using Equation 3.2 and are summarised in Table 3.1. In the following, all experimental results are reported with prototype dimensions.

3.4 RESULTS AND DISCUSSION

3.4.1 Penetration resistance profiles

The nominal penetration resistance $q_{\text{nom}}$ profiles (penetration force normalised by the maximum bearing area of the spudcan) for 15 medium dense sand overlying clay centrifuge tests are shown in Figure 3.3. They are grouped for different sand thickness. The displacement measurements are zeroed on full embedment of the bottom shoulder (i.e., spudcan embedded until a depth measured from the tip of the spigot equals $t1 + t2$ given in Table 3.1), as illustrated in Figure 3.1. In general, the potential for both punch-through and rapid leg run (see Figure 3.3) is observed in these nominal penetration resistance profiles, indicating a potential risk for spudcan installation on this type of soil stratigraphy. Punch-through and rapid leg run might occur when there is a rapid vertical spudcan displacement. For the case of punch-through, this is the result of an obvious reduction in the penetration resistance profile, whereas for rapid leg run, the rapid displacement may stem from a period of nearly constant $q_{\text{peak}}$ in the penetration resistance profile. Generally, punch-through is more likely to occur for larger $H_s/D$ ratios, and rapid leg run is more likely to occur for lower ratios of $H_s/D$. Rapid leg run is potentially just as dangerous as punch-through because the uncontrolled displacements shown here are as large as 0.3–0.7$D$.

Figure 3.4 presents selected typical nominal penetration resistance profiles for two pairs of spudcan penetration tests on very dense and medium dense sand overlying clay [noting that the $I_D = 92\%$ cases presented is based on the data of Lee (2009)]. These
profiles have been chosen because Figure 3.4(a and c) and Figure 3.4(b and d) exhibit identical $D$ values, very similar $H_s$ values, and consequently, very close values of $H_s/D$. Comparison of these penetration resistance profiles provides insight into the impact of various parameters on the peak penetration resistance and potential for hazardous failure.

![Penetration resistance profiles](image)

**Figure 3.3.** Penetration resistance profiles for 15 medium dense sand tests: (a) 6-m sand-layer thickness; (b) 5-m sand-layer thickness; (c) 3.2-m sand-layer thickness
Figure 3.4. Typical penetration resistance profiles for very dense and medium dense sand with comparable $H_s/D$ ratios (data for very dense sand from Lee 2009): (a) Test D1SP40a ($H_s/D = 0.78; I_D = 92\%$); (b) Test D1SP70a ($H_s/D = 0.44; I_D = 92\%$); (c) Test L1SP2 ($H_s/D = 0.75; I_D = 43\%$); (d) Test L1SP5 ($H_s/D = 0.43; I_D = 43\%$)

The comparisons of Figure 3.4(a and b) and Figure 3.4(c and d) demonstrate the impact of $H_s/D$ ratio on the nominal penetration resistance profiles. For both very dense and
medium dense sands, $q_{\text{peak}}$ reduces with $H_s/D$ ratio. For very dense sand with $H_s/D = 0.78$, $q_{\text{peak}}$ is 620 kPa, whereas it is 430 kPa for $H_s/D = 0.44$. The reduction in $q_{\text{peak}}$ for medium dense sand is not so obvious but still has a value of 15% for $H_s/D = 0.43$ compared with $H_s/D = 0.75$. This is so because during the mobilisation of $q_{\text{peak}}$, for high $H_s/D$ ratios, the influence zone is confined mainly to the sand layer, which would contribute to a large $q_{\text{peak}}$. For the very dense sand tests in Figure 3.4(a and b), the reduction in $H_s/D$ ratio decreases the magnitude of the peak penetration resistance $q_{\text{peak}}$ and also the potential length of the uncontrolled vertical displacement. For the medium dense sand tests in Figure 3.4(c and d), the impact of reducing the $H_s/D$ ratio is to change the potential for failure from punch-through to rapid leg run. This is further confirmed by the other medium dense sand tests presented in Figure 3.3.

Comparison of Figure 3.4(a and c) and Figure 3.4(b and d) highlights the impact of $I_D$ on the failure mode. For high $H_s/D$ ratios, in Figure 3.4(a and c), reducing the $I_D$ changes the penetration resistance from a peaked and sudden failure, with significant rapid post-peak resistance reduction, to a more progressive failure with attenuated post peak resistance reduction. For low $H_s/D$ ratios, in Figure 3.4(b and d), the same trend is evident except that the medium dense sand failure potential becomes a rapid leg run rather than punch-through.

This indicates that punch-through or rapid-leg-run failure is a potential problem for sand overlying clay stratigraphies involving sand from medium dense to very dense states because even in Figure 3.4(d), which exhibits the smallest $q_{\text{peak}}$, the potential for uncontrolled rapid leg run was observed. As a result, the failure-stress-dependent model proposed by Lee et al. (2013b) for very dense sand overlying clay is developed in this
paper to accurately predict $q_{\text{peak}}$ for problems involving sand from medium dense to very dense states.

### 3.4.2 Peak penetration resistance

Figure 3.5 presents the peak penetration resistance $q_{\text{peak}}$ versus the widest cross-sectional area for each of the spudcans tested. The data are grouped in accordance with the thickness and relative density of the sand layer. The figure illustrates that the general variations in the peak penetration resistances with the widest cross-sectional areas may be fitted with power law equations. Specific equations are not given for these fits because they are only intended to demonstrate trends in the results.

![Figure 3.5. Peak penetration resistance as a function of the wisest cross-sectional area of spudcan [data with $I_D = 92\%$ from Lee (2009)]](image)

The bearing pressures applied by spudcans for modern jack-ups are reported to be in the range of 200-600 kPa (Osborne et al. 2008). The experimentally measured peak penetration resistances shown in Figure 3.5 are within this range, which suggests that the current centrifuge model tests were appropriately scaled to calibrate the proposed failure-stress-dependent model.
3.4.3 Depth of peak penetration resistance

In addition to the peak penetration resistance $q_{\text{peak}}$, the depth at which the peak penetration resistance occurs must be predicted because both are necessary for providing a full penetration resistance profile for spudcan penetration on sand overlying clay. Based on centrifuge tests on dense and very dense sand overlying clay conducted at UWA and at the National University of Singapore (NUS), Teh et al. (2010) proposed that the effective sand thickness $H_{\text{eff}}$ at mobilisation of $q_{\text{peak}}$ was equal to $0.88H_s$. The depth of penetration required to mobilise the peak penetration resistance $d_{\text{peak}}$ ($d_{\text{peak}} = H_s - H_{\text{eff}}$) is therefore $0.12H_s$.

![Graph showing correlation between $H_{\text{eff}}/D$ and $H_s/D$](image)

**Figure 3.6. Correlation between $H_{\text{eff}}/D$ and the normalised sand thickness $H_s/D$**

Figure 3.6 is a summary of the correlation between $H_{\text{eff}}/D$ and $H_s/D$ ratio for all the centrifuge tests listed in Tables 3.1 and 3.2 except the beam centrifuge test of Lee (2009), for which the penetration resistance profiles were not available. The relationship proposed by Teh et al. (2010) is consistent for both UWA and NUS geometries of spudcans and the soil properties reported in Tables 3.1 and 3.2. Thus the depth of peak
penetration resistance relative to the lowest elevation of the spudcan’s widest cross-sectional area may be expressed with confidence as:

\[ \frac{d_{\text{peak}}}{0.12} = H_s \]  

\[ (3.3) \]

Table 3.2. Relevant parameters for centrifuge tests from the literature used in the comparative prediction performance study

<table>
<thead>
<tr>
<th>Investigation</th>
<th>Test Name</th>
<th>Geometry</th>
<th>Sand</th>
<th>Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( H_s ) (m)</td>
<td>( D ) (m)</td>
<td>( H/D )</td>
</tr>
<tr>
<td>Teh (2007) NUS Tests</td>
<td>NUS_F1</td>
<td>3.0</td>
<td>10</td>
<td>0.30</td>
</tr>
<tr>
<td></td>
<td>NUS_F2</td>
<td>5.0</td>
<td>10</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td>NUS_F3</td>
<td>7.0</td>
<td>10</td>
<td>0.70</td>
</tr>
<tr>
<td></td>
<td>NUS_F4</td>
<td>7.7</td>
<td>10</td>
<td>0.77</td>
</tr>
<tr>
<td></td>
<td>NUS_F5</td>
<td>10.0</td>
<td>10</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>NUS_F8</td>
<td>5.0</td>
<td>10</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td>NUS_F9</td>
<td>7.0</td>
<td>10</td>
<td>0.70</td>
</tr>
<tr>
<td>Teh (2007) UWA Tests</td>
<td>UWA_F3</td>
<td>3.5</td>
<td>4</td>
<td>0.88</td>
</tr>
<tr>
<td></td>
<td>UWA_F4</td>
<td>3.5</td>
<td>6</td>
<td>0.58</td>
</tr>
<tr>
<td></td>
<td>UWA_F10</td>
<td>7.1</td>
<td>8</td>
<td>0.89</td>
</tr>
<tr>
<td>Lee (2009) Drum Centrifuge Tests</td>
<td>D1SP40a</td>
<td>6.2</td>
<td>8</td>
<td>0.78</td>
</tr>
<tr>
<td></td>
<td>D1SP50a</td>
<td>6.2</td>
<td>10</td>
<td>0.62</td>
</tr>
<tr>
<td></td>
<td>D1SP60a</td>
<td>6.2</td>
<td>12</td>
<td>0.52</td>
</tr>
<tr>
<td></td>
<td>D1SP70a</td>
<td>6.2</td>
<td>14</td>
<td>0.44</td>
</tr>
<tr>
<td></td>
<td>D1SP80a</td>
<td>6.2</td>
<td>16</td>
<td>0.39</td>
</tr>
<tr>
<td>Lee (2009) Beam Centrifuge Tests</td>
<td>B1S7SP8a</td>
<td>7.0</td>
<td>8</td>
<td>0.88</td>
</tr>
<tr>
<td></td>
<td>B1S7SP8b</td>
<td>7.0</td>
<td>8</td>
<td>0.88</td>
</tr>
<tr>
<td></td>
<td>B1S7SP8c</td>
<td>7.0</td>
<td>8</td>
<td>0.88</td>
</tr>
<tr>
<td></td>
<td>B1S7SP14a</td>
<td>7.0</td>
<td>14</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td>B1S7SP14b</td>
<td>7.0</td>
<td>14</td>
<td>0.50</td>
</tr>
</tbody>
</table>
3.5 FAILURE-STRESS-DEPENDENT PREDICTION MODEL

3.5.1 Performance of original failure-stress-dependent model

As shown in Figure 3.7, Lee et al. (2013b) proposed an analytical model that assumed that the peak penetration resistance occurs when a sand frustrum with a dispersion angle (the angle between the assumed slip surface and the vertical plane) equal to the angle of dilation \( \psi \) is pushed into the underlying clay. Hence \( q_{\text{peak}} \) is the sum of the frictional resistance in the sand, the bearing capacity of the underlying clay, and the weight of the sand frustrum. The operative friction and dilation angles are related to \( q_{\text{peak}} \) using a modified form of Bolton’s (1986) empirical relationships, i.e.,

\[
I_R = I_D (Q - \ln(p')) - 1 \quad 0 < I_R < 4 \quad (3.4)
\]

\[
\phi' - \phi_{cv} = mI_R \quad (3.5)
\]

\[
0.8\psi = \phi' - \phi_{cv} \quad (3.6)
\]

where \( I_R \) = dilatancy indicator in degrees; \( Q \) = natural logarithm of the grain crushing strength expressed in kilopascals; \( p' \) = mean effective stress; \( \phi' \) = operative friction angle; \( \phi_{cv} \) = critical-state friction angle; and \( m \) = a constant. Lee et al. (2013a) used 25 centrifuge model tests on very dense sand overlying clay and accompanying small-strain finite-element simulations to back-analyse a best-fit value for \( m \) of 2.65. The authors also performed similar simulations of the medium dense sand overlying clay tests presented in this paper with good comparability with the experimental measurements, providing confidence that a value for \( m \) of 2.65 was appropriate irrespective of the relative density of the sand layer.

In Lee’s analytical model, a distribution factor \( D_F \) is defined to relate the local stress along the failure surface to the average vertical effective stress, more specifically, the
ratio of the vertical effective stress at the slip surface to the mean vertical effective stress. The distribution factor depends on the $H/D$ ratio of flat or spudcan foundations, and bilinear equations were proposed to depict the preceding relationship.

\[
q_{\text{peak}} = \frac{q_0}{D} \quad \text{Sand: } \phi', \gamma's \\
q_{\text{clay}} = \frac{q_{\text{clay}}}{H_s} \quad \text{Clay: } s_u, k
\]

**Figure 3.7. Failure-stress-dependent mechanism (data from Lee et al. 2013b)**

For current tests for medium dense sand layers, optimized $D_F$ values were derived by varying $D_F$ until the $q_{\text{peak}}$ predicted by the original model of Lee et al. (2013b) was equal to the measured value. The optimized $D_F$ values are plotted against $H_s/D$ ratio in Figure 3.8(a) alongside the calculated resistance divided by the experimental resistance ($q_{\text{peak, calculated}}/q_{\text{peak, measured}}$) in Figure 3.8(b). The bilinear variations in $D_F$ with $H_s/D$ ratio suggested by Lee et al. (2013b) are plotted in Figure 3.8 as well. Some skew of the regression line is evident in Figure 3.8(b), particularly with smaller $H_s/D$ ratios, which means that the original model underestimates the peak penetration resistance for smaller $H_s/D$ ratios. This is further verified by Figure 3.8(a) because the bilinear $D_F$ equations do not capture the trend well, especially for cases where $H_s/D < 0.3$. This is not
unexpected because the preceding failure mechanism and bilinear $D_F$ relationships in their original state were calibrated solely using experimental data for spudcans in a single height ($H_s = 6.2$ m) of very dense sand ($I_D = 92\%$) overlying clay. The original mechanism was validated, and not calibrated, using data from only three tests performed on loose and medium dense sand overlying clay reported in the literature. In addition, the embedment depth achieved during mobilisation of $q_{\text{peak}}$ is not accounted for.

![Figure 3.8. Performance of Lee’s failure-stress-dependent model: (a) calibrated $D_F$ and bilinear relationship equations; (b) predictions based on the model](image)

3.5.2 Modification of the failure-stress-dependent model

To account for the embedment depth at failure, the original mechanism was modified as shown in Figure 3.9. In this modified failure mechanism, the peak penetration resistance is derived following the same procedure as in Lee et al. (2013b), but the embedment depth attained during mobilisation of $q_{\text{peak}}$ is taken into account. The $D_F$ values are optimised, and a new power relationship with $H_s/D$ ratio is proposed based on the modified failure mechanism. In brief, the problem is treated mathematically as a series of infinitesimally thin horizontal disks, which allows the following differential equation to be formulated:
\[
\frac{\partial \sigma^*_z}{\partial z} + \frac{E^* \tan \psi}{(D/2 + z \tan \psi)} \sigma^*_z - \gamma_s' = 0
\]  
(3.7)

where \(\sigma^*_z\) = average vertical effective stress in each horizontal disk at depth \(z\). The parameter \(E^*\) is adopted to simplify the algebra and taken as:

\[
E^* = 2 \left[1 + D_F \left(\frac{\tan \phi^*}{\tan \psi} - 1\right)\right]
\]  
(3.8)

where \(\phi^*\) = reduced friction angle caused by non-associated flow, which can be expressed as (Drescher and Detournay 1993):

\[
\tan \phi^* = \frac{\sin \phi \cos \psi}{1 - \sin \phi \sin \psi}
\]  
(3.9)

In reality, \(D_F\) probably varies with depth, and it is very difficult to derive an accurate expression of \(D_F\) with various combinations of soil strength and \(H_s/D\). In order to integrate the differential force equilibrium Equation 3.7 and obtain a simplified equation for practical design, it is assumed \(D_F\) is a constant, thus Equation 3.7 can be integrated to give:

\[
\left(\frac{D}{2} + z \tan \psi\right)^{E^*} \sigma^*_z = \gamma_s' \left(\frac{D}{2} + z \tan \psi\right)^{E^*+1} + C
\]  
(3.10)

where \(C\) = a constant that can be determined through the following critical condition. Referring to the conceptual model in Figure 3.9, when the depth \(z\) is equal to the effective sand thickness \(H_{eff}\), the mean vertical effective stress is equal to the bearing capacity of the underlying clay layer; thus \(C\) can be expressed as:

\[
C = \left(\frac{D}{2} + H_{eff} \tan \psi\right)^{E^*} \left[\left(N_{c0} s_{um} + q_0 + \gamma_s' H_{eff} + \gamma_s' d_{peak}\right) \left(\frac{D}{2} + H_{eff} \tan \psi\right)^{E^*+1}\right]\left(\frac{D}{2} + H_{eff} \tan \psi\right)^{E^*+1}\right]
\]  
(3.11)
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where \( N_{c0} \) = bearing capacity factor of clay at foundation base, which is obtained using the relationship proposed by Houlsby and Martin (2003) for circular foundations with shear strength increasing linearly with depth; \( s_{um} \) = undrained strength of the clay at the sand-clay interface; and \( q_0 \) = effective overburden pressure at the depth of the foundation.

By substituting Equation 3.11 into Equation 3.10, the mean vertical effective stress can be expressed as:

\[
\frac{\gamma'_{s} \left( \frac{D}{2} + z \tan \psi \right)}{\tan \psi (E^* + 1)} + \left( \frac{D}{2} + H_{\text{eff}} \tan \psi \right)^{E^*} \left[ \left( N_{c0} s_{um} + q_0 + \gamma'_{s} H_{\text{eff}} + \gamma'_{s} q_{\text{peak}} \right) \frac{\gamma'_{s} \left( \frac{D}{2} + H_{\text{eff}} \tan \psi \right)}{\tan \psi (E^* + 1)} \right]
\]

\[(3.12)\]

According to Figure 3.9, the spudcan penetration depth \( z \) is measured from the depth where the peak penetration resistance occurs, and the peak penetration resistance can be obtained by setting \( z \) equal to zero. As discussed earlier, \( H_{\text{eff}} = 0.88H_s \) for both tests involving very dense, dense and medium dense sand layers, and by substituting \( \sigma'_{z} \) with \( q_{\text{peak}} \) and inputting the values for \( d_{\text{peak}} \) and \( H_{\text{eff}} \) into Equation 3.12, the peak penetration resistance is thus expressed in terms of \( H_s \):

\[
q_{\text{peak}} = \left( N_{c0} s_{um} + q_0 + 0.12 \gamma'_{s} H_s \left( 1 + \frac{1.76H_s}{D} \tan \psi \right)^{E^*} \right) + \gamma'_{s} \frac{D}{2\tan \psi (E^* + 1)} \left[ 1 - \left( 1 - \frac{1.76H_s}{D} E^* \tan \psi \right) \left( 1 + \frac{1.76H_s}{D} \tan \psi \right)^{E^*} \right]
\]

\[(3.13)\]

for cases where \( \phi' > \phi_{cv} \). Similarly, for cases where \( \phi' = \phi_{cv} \), the peak penetration resistance can be calculated as:
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\[ q_{\text{peak}} = \left( N_{\text{eo}} s_{\text{um}} + q_0 + 0.12 \gamma'_s H_s \right) e^{E_0} + 0.88 \gamma'_s H_s \left[ e^{E_0} \left( 1 - \frac{1}{E_0} \right) + \frac{1}{E_0} \right] \]  

(3.14)

where \( E_0 \) is equal to:

\[ E_0 = 3.52 D_k \sin \phi'_s \frac{H_s}{D} \]  

(3.15)

In Equation 3.4, the mean effective stress \( p' \) is substituted with \( q_{\text{peak}} \) in Equation 3.13, and through an iterative procedure using Equations 3.4–3.6 and 3.13 in a spreadsheet analysis (e.g., in Microsoft Excel), this approach allows the operative friction angle and dilatancy (and thus dispersion angle of the sand frustum) to be related to the stress level at failure rather than to the initial state. This method is advantageous because it avoids the use of design charts, and sensitivity analysis can be conducted with ease.

Figure 3.9. Modified failure-stress-dependent mechanism accounting for embedment depth attained during mobilisation of \( q_{\text{peak}} \)

Based on the modified failure-stress-dependent model and with the benefit of the additional experimental data presented, the distribution factor was recalibrated and optimised in terms of a wide range of spudcan geometries and soil conditions incorporating both medium dense and very dense sand overlying clay. The optimised \( D_F \)
for the additional 15 centrifuge tests for medium dense sand and tests for the very dense sand by Lee (2009) are presented in Figure 3.10(a). It is observed that the relationship between $D_F$ and $H_s/D$ ratio is nonlinear and better fitted with the power law:

$$D_F = 0.642 \left( \frac{H_s}{D} \right)^{-0.576} \quad \text{as} \quad 0.16 \leq \frac{H}{D} \leq 1.0 \quad (3.16)$$

Using the original model and the linear relationships proposed by Lee et al. (2013b), the coefficient of determination $R^2$ is 0.77 for the experimental data in Figure 3.10, whereas $R^2 = 0.94$ using the modified model and the power relationship in Equation 3.16. For low $H_s/D$ ratios, the embedded volume of the spudcan during mobilisation of $q_{peak}$ is a far larger proportion of the volume of the inverted truncated cone in the modified failure mechanism than that for high $H_s/D$ ratios. This embedded volume causes increasing lateral stress and then the larger values of $D_F$ (even greater than unity for $H_s/D < 0.4$).

The embedded volume of the spudcan also can be expressed in terms of $H_s/D$ ratio using a similar power relationship. Although it is not possible to link $D_F$ directly to the embedded volume of the spudcan at $q_{peak}$ (given that the increase in mean stress at the failure surface of the proposed failure mechanism is highly unlikely to be directly proportional to the volume of sand displaced during spudcan embedment), it is logical to use similar power relationship proposed in Equation 3.16 to describe $D_F$.

Similar to Figure 3.8(b), the scattered markers in Figure 3.10(b) show the predictions for both medium dense and very dense sand tests tested on the modified mechanism and power relationship of $D_F$. There is reduced skew for the whole range of $H_s/D$ ratio of practical interest. This is so because the embedment depth during the mobilisation of peak penetration resistance is accounted for in the modified failure mechanism, and the new $D_F$ relationship is calibrated for a larger range of spudcan diameters and soil
properties. More tests are needed to investigate the suitability of the $D_F$ relationship for different spudcan shapes (conical angles).

Figure 3.10. Performance of modified failure-stress-dependent model: (a) calibrated $D_F$ and nonlinear relationship equation; (b) predictions based on the modified model

3.5.3 Performance of the modified model in predicting $q_{peak}$

To further validate the performance of the modified and recalibrated model, three series of additional centrifuge test data for the penetration of spudcans into sand overlying normally or lightly over-consolidated clay have been compared. These additional tests consist of seven tests by Teh (2007) in the NUS beam centrifuge, three tests by Teh (2007) in the UWA beam centrifuge, and five tests by Lee (2009) in the UWA beam centrifuge. Tables 3.1 and 3.2 contain a summary of the relevant geometric and material parameters of these tests, as well as the experimental results. All the data used in this validation were derived from tests with siliceous sand; consequently, $Q$ in Equation 3.4 was assumed to be 10 (Bolton 1986). The critical-state friction angle was taken as 31° for the sand used in the UWA tests (White et al. 2008) and 32° for the Toyoura sand used in the NUS tests (Jamiolkowski et al. 2003).
The performance of the modified model is compared with that of the primary model (based on the load spread method) and alternative (based on the punching shear method) recommendations for the ISO (2012). For each method, the measured and calculated peak penetration resistance predictions are presented with \( q_{\text{peak, calculated}} \) against \( q_{\text{peak, measured}} \) and \( q_{\text{peak, calculated}}/q_{\text{peak, measured}} \) against \( H_s/D \) ratio, as shown in Figure 3.11. A linear regression line is used in the \( q_{\text{peak, calculated}}/q_{\text{peak, measured}} \) figure to identify any apparent trend in performance of the calculation with respect to \( H_s/D \) ratio. Table 3.3 provides a summary of key performance indicators such as mean, maximum, minimum, \( \sigma \), and skew angle, \( \theta^* \), where \( \theta^* = \arctan(s) \), with \( s \) being the slope of the regression line.

For both the ISO (2012) primary and alternative recommendations, conservative predictions of \( q_{\text{peak}} \) are obtained, with most of the predicted peak penetration resistances less than 60% of the measured peak penetration resistances. The reason is that both methods ignore the properties of the sand: the load spread factor is not related to the sand properties in the primary recommendation, and the frictional resistance through the sand is expressed in terms of the normalised shear strength of the underlying clay layer in the alternative recommendation. The alternative recommendation made by ISO (2012) based on the punching shear mechanism actually performs better than the primary recommendation based on the load spread method. Referring back to Figure 3.4 demonstrates the implication of this conservative prediction of \( q_{\text{peak}} \). For both medium dense and very dense sands, such conservative would lead to gross underestimation of the potential impact of both punch-through and rapid leg run because the depth over which the event may occur also would be underestimated. The skew angles presented in Figure 3.11 and Table 3.3 demonstrate that the methods based on ISO (2012) recommendations exhibit significant bias in performance, with worsening predictions for larger \( H_s/D \) ratios for the load spread method. This is so because at higher \( H_s/D \)
ratios the thicker sand layer leads to the capacity of resistance generated by the sand being a larger proportion of $q_{\text{peak}}$. Hence, by ignoring the contribution of the sand layer in the load spread method, the predictions worsen with increasing $H_s/D$ ratio.

![Graph showing comparison of measured and calculated $q_{\text{peak}}$](image)

**Figure 3.11.** Comparison of measured and calculated $q_{\text{peak}}$ using ISO (2012) recommendations and modified and recalibrated failure-stress-dependent model ($h$ = horizontal; $v$ = vertical)

**Table 3.3. Summary of Model Performance Indicators for Each of the Prediction Methods**

<table>
<thead>
<tr>
<th>Method</th>
<th>$q_{\text{peak, calculated}}/q_{\text{peak, measured}}$</th>
<th>Min.</th>
<th>Max.</th>
<th>Mean</th>
<th>$\sigma$</th>
<th>$\theta^*$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ISO (2012) Recommendations (Load Spread Method with 1h:3v Spread Ratio)</td>
<td></td>
<td>0.07</td>
<td>0.41</td>
<td>0.23</td>
<td>0.09</td>
<td>-13.39</td>
</tr>
<tr>
<td>ISO (2012) Recommendations (Load Spread Method with 1h:5v Spread Ratio)</td>
<td></td>
<td>0.05</td>
<td>0.37</td>
<td>0.18</td>
<td>0.09</td>
<td>-14.30</td>
</tr>
<tr>
<td>ISO (2012) Alternative Recommendations (Punching Shear Method)</td>
<td></td>
<td>0.38</td>
<td>0.78</td>
<td>0.50</td>
<td>0.09</td>
<td>17.66</td>
</tr>
<tr>
<td>Modified Failure Stress Dependent Model with Recalibrated $D_f$</td>
<td></td>
<td>0.83</td>
<td>1.39</td>
<td>1.01</td>
<td>0.11</td>
<td>2.27</td>
</tr>
</tbody>
</table>
In contrast to the ISO (2012) recommendations, the modified failure-stress-dependent model with the recalibrated $D_F$ provides an average of all the predictions of $q_{\text{peak}}$, calculated/$q_{\text{peak}}$, measured of 1.01 and a $\sigma$ of 0.11. All predictions, apart from test NUS_F5 from Teh (2007), are within ±20% of the measured values. The significantly less skew with respect to $H_s/D$ ratio for the predictions indicates that the modified failure-stress-dependent model is capable of predicting penetration response in sand overlying clay far more effectively than the ISO methods. This is so because the failure stress is highly dependent on sand thickness and foundation size, as demonstrated in Figure 3.5, and the operative friction angle and dilation angle of inclination of the inverted truncated-cone mechanism is related to the failure stress in the modified failure-stress-dependent model.

By adopting the modified failure-stress-dependent model to improve the accuracy of estimation of $q_{\text{peak}}$, the uncertainty and risk associated with spudcan installation in sand overlying clay stratigraphies may be reduced.

### 3.6 CONCLUSIONS

Fifteen centrifuge tests have been conducted within a drum centrifuge to investigate spudcan foundation behaviour on medium dense sand overlying clay, representing the first comprehensive investigation of punch-through or rapid-leg-run potential for medium dense sand overlying clay. The tests covered different prototype sand thickness in the range 3.2-6 m and spudcan diameters in the range 6-20 m, corresponding to $H_s/D$ ratios of 0.16 to 1. This covers the range of practical interest for punch-through failure of jack-up platforms. These new data were combined with the data for spudcans penetrating very dense sand overlying clay from Lee (2009) to allow recalibration of the modified failure-stress-dependent model. Interpretation of these data has led to the following conclusions:
1. The potential for catastrophic punch-through and rapid leg run of spudcans, already demonstrated for very dense sand overlying clay, is also a potential problem for medium dense sand overlying clay sites.

2. The depth of occurrence of $q_{\text{peak}}$ has been further confirmed experimentally to be a function of $H_s$ for the range of $H_s/D$ ratios of practical interest. Coupling the depth of occurrence with accurate prediction of $q_{\text{peak}}$ provides the first step in predicting the risk of punch-through failure for sand overlying clay sites.

3. The failure-stress-dependent model of Lee et al. (2013b) for predicting $q_{\text{peak}}$ on sand overlying clay has been modified to account for mobilisation-induced embedment. This modified mechanism has been used to derive an equation to describe $q_{\text{peak}}$ in terms of the undisturbed sand thickness.

4. A new relationship has been proposed for $D_F$ that is used to relate the stress at the failure surface to the average vertical effective stress in the modified failure-stress-dependent model. This provides improved prediction of $q_{\text{peak}}$ over a larger range of sand-thickness-to-spudcan-diameter ($H_s/D$) ratios as well as sand relative densities. At this juncture, the new relationship has been calibrated and validated only for spudcan shapes similar to that tested here.

5. The modified failure-stress-dependent model, which is based on a kinematically admissible failure mechanism and accounts for the embedment depth caused by mobilisation of $q_{\text{peak}}$, was shown to be capable of accurately predicting the peak penetration resistance $q_{\text{peak}}$ for both medium dense and very dense sand overlying clay.
6. The performance of the modified failure-stress-dependent model has been compared with the current recommended practice based on ISO (2012) primary and alternative recommendations, and it was demonstrated that the modified model provides more accurate prediction of $q_{\text{peak}}$ with less bias in relation to $H_s/D$ ratios for a wide range of medium dense to very dense sand over clay sites.

In summary, when used to back-calculate centrifuge data, ISO (2012) prediction methods significantly underestimate $q_{\text{peak}}$ during spudcan penetration on sand overlying clay. The modified failure-stress-dependent model improves the $q_{\text{peak}}$ predictions, and its adoption for field conditions offshore has the potential to better predict the potential for punch-through or rapid-leg-run events during spudcan installation on sand overlying clay. This would significantly reduce the risk associated with operating jack-up platforms in offshore locations with sand overlying clay soil stratigraphy.

### 3.7 REFERENCES


Cheong, J. (2002). “Physical testing of jack-up footings on sand subjected to torsion.” Honours thesis, Univ. of Western Australia, Perth, Australia.


Chapter 3: Predicting peak resistance of spudcan penetrating sand overlying clay through centrifuge model testing


CHAPTER 4. PREDICTING THE RESISTANCE PROFILE OF A SPUDCAN PENETRATING SAND OVERLYING CLAY

ABSTRACT
Assessment of the risk of punch-through failure of spudcan foundations on sand overlying clay requires prediction of the full penetration-resistance profile, from touchdown and through punch-through to equilibrium of the vertical resistance at depth in the underlying clay layer. This study uses the Coupled Eulerian-Lagrangian approach, a large deformation finite element analysis method, to model the complete penetration resistance profile of a spudcan on sand overlying clay. The sand is modelled using the Mohr-Coulomb model, while the clay is modelled using a modified Tresca model to account for strain softening. The numerical method is then used to simulate a series of spudcan penetration tests, performed in a geotechnical centrifuge, on medium dense sand overlying clay. The punch-through behaviour observed in the experiments is replicated, and the penetration resistance profiles from numerical analyses are generally a reasonable match to the experimental measurements. The influences of the sand layer height to foundation diameter ratio, sand-clay interface shear strength and strength gradient in clay on the penetration resistance profiles are explored in a complementary parametric study. The penetration resistance in the underlying clay layer is well predicted using a simple linear expression for the bearing capacity factor for the spudcan and underlying sand plug. This expression is combined with an existing failure stress-dependent model for predicting peak resistance to form a simplified method for prediction of the full penetration resistance profile. This new method provides estimates of the vertical penetration that the spudcan will run during the punch-through event. It is
validated against centrifuge model tests of spudcans in both medium dense and very dense sand overlying clay soil profiles.

4.1 INTRODUCTION

Unexpectedly sudden and rapid penetrations of spudcan foundations during installation are a major risk to the stability of jack-up platforms. This can occur when under vertical loading the large (often ~ 20 m diameter) spudcan pushes a layer of strong sandy material into an underlying weak clay layer. Rapid spudcan and leg penetration occurs until the applied jack-up weight (and preloading ballast weight) equates with the resistance in the underlying clay layer. If not handled properly, the large displacement of the leg may cause damage to the structure of the jack-up or down-time. This type of event is termed a punch-through failure.

To assess the risk of punch-through during the installation of jack-up rigs, an accurate prediction of the spudcan penetration resistance profile is thus critical. Two ‘wished-in-place’ methods are recommended by the ISO (2012) guidelines to estimate the peak penetration resistance of a spudcan in layered strata where punch-through might be a possibility. In both methods the bearing capacity is calculated at a specified penetration depth, assuming a pre-embedded spudcan and undisturbed soil stratigraphy and strengths. When used to back-calculate centrifuge data, the ISO (2012) prediction methods significantly underestimate the potential for punch-through (Hu et al. 2014). There are currently no guidelines for the calculation of the bearing capacity following punch-through into the underlying clay layer that account for the combined bearing capacity of the spudcan and any sand trapped beneath, as observed by Teh et al. (2008) using particle image velocimetry (PIV) analysis.
Simple methods are needed to predict the full penetration resistance profile for a spudcan penetrating through sand overlying clay that can be used routinely in site-specific assessment of jack-up safety (Osborne et al. 2006). Such methods should account for the mechanistic changes that occur at different stages of penetration, as observed by Teh et al. (2008). For simplicity, two key events might be used to define the severity of a punch-through event: (i) the magnitude and depth of peak resistance in the sand layer and (ii) the depth at which the resistance in the underlying clay layer becomes equal to the peak resistance. The depth between these two events is thus an estimate of the plunge depth that may be experienced in the field during preloading (though it is acknowledged that the load on the leg will reduce due to the buoyancy force from the immersion of the hull into the water) and can be used to check against serviceability limits for the jack-up platform. For the measurement of plunge depth from the testing profile, a tangent line from the depth at which the spigot is fully embedded following the penetration resistance profile towards peak resistance is constructed, followed by a vertical line to the equilibrium depth in the clay layer (where the current resistance is equal to the peak resistance in sand). The punch-through distance, $d_{\text{punch}}$, is the length of the line from the depth of peak resistance until vertical equilibrium is re-established.

The magnitude and depth of the peak resistance in the sand layer is readily calculable using the failure stress-dependent model derived by Lee et al. (2013b) for very dense sand overlying clay. This model assumes that an inverted and truncated cone of sand is pushed down into the underlying clay, mobilising shearing around the periphery of the inverted truncated cone and clay-bearing capacity at the base. Stress-dependent dilatancy is incorporated into the model using a modified form of Bolton’s correlations
by relating the dilatancy to the bearing pressure at failure (Bolton 1986). The form of this model was enhanced by Hu et al. (2014) to account for mobilisation embedment depth and further validated experimentally for medium dense to very dense sand states.

Calculating the bearing capacity in the underlying clay is complicated by sand trapped underneath the spudcan. The bearing pressure of a spudcan in clay is commonly expressed as:

\[ q_{clay} = N_c s_{u0} + \frac{V_f \gamma_c'}{A} \]  

(4.1)

where \( N_c \) is the bearing capacity factor; \( s_{u0} \) is the intact soil strength at the lowest elevation of the spudcan widest cross-sectional area (termed Load Reference Point, LRP); \( V_f \) is the embedded foundation volume below the spudcan LRP; \( \gamma_c' \) is the effective unit weight of the clay and \( A \) is the nominal surface area of the foundation (\( A = \pi D^2/4 \) and \( D \) is the diameter of the spudcan). The first term is the bearing capacity for a weightless soil, while the second term accounts for overburden pressure. Following punch-through on sand overlying clay, the bearing capacity factor \( N_c \) and volume of the foundation \( V_f \) should be related to the composite geometry of the foundation and the trapped sand plug. However, application of Equation 4.1 is difficult for this scenario due to uncertainty about the trapped sand plug volume and geometry. For example, Teh et al. (2008) observed using PIV analyses that the upper portion of the trapped sand beneath the spudcan resembled a cylinder with diameter similar to the spudcan, whereas the shape of the lower part was slightly irregular due to the meta-stable state of the trapped sand. In the experiments of Teh et al. (2008), the trapped sand plug height, \( H_{plug} \), was \(~1H_s\) where \( H_s \) is the height of the sand layer. Lee (2009) performed a series of small strain numerical analyses that assumed that the spudcan and sand plug could be idealised as a composite cylindrical foundation. Expressions for \( N_c \) were related to the
assumed composite foundation height and diameter, facilitating prediction of the full penetration resistance profile from a measured soil shear strength profile. In contrast to Teh et al. (2008), Lee (2009) found that a range of $H_{plug}$ of 0.6 to 0.9$H_s$ was required to attain a good fit to the experimental measurements. Such uncertainty limits confidence in the application of Equation 4.1 and further verification is necessary. An alternative approach is to back-calculate the bearing capacity factors as $N_c = q/s_u$ from the experimental measurements of full spudcan load-penetration tests, as performed by Lee et al. (2013a), resulting in:

$$N_c = 14 \frac{H_s}{D} + 9.5 \quad \left(0.21 \leq \frac{H_s}{D} \leq 1.12\right) \quad (4.2)$$

It should be noted here that in this way the shaft friction acting on the side of the sand plug in clay has been considered implicitly as it is not easy to calculate the shaft friction force during the continuous penetration of the spudcan in the underlying clay layer. The full penetration resistance profile can also be predicted using large deformation finite element methods. These methods require no prior assumption of the failure mechanisms and may be in conjunction with advanced constitutive models to attempt to faithfully replicate soil behaviour. Compared with traditional Lagrangian finite element methods, large deformation analysis avoids severe mesh distortion. Two such methods have been used to replicate the continuous penetration process of a spudcan on sand overlying clay: (i) The Remeshing and Interpolation Technique with Small Strain (RITSS) method was used by Yu et al. (2012) to investigate the effects of the undrained shear strength of the underlying clay and the thickness and friction angle of the upper sand. A simple equation was proposed to relate the post-peak bearing capacity factor to the soil properties in which the bearing capacity factor is directly related to the strength ratio between the sand and the clay:
\[ N_c = 3.3 \left( \frac{H_s \gamma'_s \tan \phi'}{s_{u0}} \right) + 12 \]  

(4.3)

where \( \gamma'_s \) and \( \phi' \) are the effective unit weight and internal friction angle of sand, respectively. However, this equation is summarised based on a limited numerical study and does not account for the increase of \( N_c \) with \( H_s / D \) observed experimentally by Lee et al. (2013a).

(ii) The Coupled Eulerian-Lagrangian (CEL) approach that is available in the commercial package ABAQUS/Explicit. The CEL approach was first adopted for research of spudcan performance by Tho et al. (2009, 2012) to compare numerical penetration resistances and soil flow mechanisms with centrifuge test results of spudcan penetration in uniform clay, normally consolidated clay as well as in layered soil profiles involving stiff clay/sand overlying soft clay. The Mohr-Coulomb model and a strength degradation model were used to describe the behaviour of sand and clay layers, respectively. Qiu and Henke (2011) and Qiu and Grabe (2012) later incorporated hypoplastic and visco-hypoplastic constitutive models into the CEL approach to better simulate the complex rate and strain-hardening-softening dependent behaviours of sand and clay. The spudcan penetration behaviour in loose sand overlying weak clay was investigated in Qiu and Henke (2011), while the influences of the relative density of sand and spudcan diameter on the penetration resistance and failure mechanism when spudcan penetrating sand overlying clay were discussed in Qiu and Grabe (2012). No recommendations on the bearing capacity factor for the composite spudcan and sand plug in the underlying clay were provided in previous CEL studies.

In this paper, the CEL approach is used to simulate continuous spudcan penetration on sand overlying clay with increasing shear strength with depth, while accounting for
strain softening in the clay layer. The numerical methods are first validated by
comparison with previous numerical and centrifuge test data for spudcan penetration in
normally consolidated (NC) clay and in uniform sand. The impacts of mesh density,
penetration rate, foundation interface friction and clay softening are then considered.
The CEL analyses are conducted to simulate a series of centrifuge tests performed on
medium dense sand overlying clay alongside a parametric study that broadens the scope
to cover the geometric conditions relevant to spudcan punch-through in the field. Sand
plug heights and bearing capacity factors in the clay layer are then inferred from the
CEL analyses and are shown to be a good fit for a simple linear relationship. These
simple equations can be used to predict the bearing capacity profile in the lower clay
layer and, when coupled with the failure stress-dependent model by Hu et al. (2014), to
derive an estimate for the depth of a punch-through event. The performance of the
simplified prediction method is verified against experimental centrifuge data for
medium dense to very dense sand overlying clay.

4.2 NUMERICAL METHODOLOGY

4.2.1 Implementation of CEL approach

In the CEL approach, the spudcan and soil are discretised using Lagrangian and
Eulerian mesh respectively. The Eulerian mesh is composed of 8-node linear
hexahedron elements (an integration point per element) with reduced integration and
hourglass control. The soil materials are allowed to flow through Eulerian elements
whose nodes have fixed locations (Dassault Systèmes, 2011). The Eulerian mesh is
initially composed of two parts, one that is initially occupied by soil and another that is
void to accommodate any soil heave created during the penetration process. An Eulerian
element may be partially void or filled with multiple materials. The presence and
volume fractions of different materials in each Eulerian element are specified at the beginning of the analysis. In contrast, only one material is contained in a Lagrangian element. During the analysis, the Eulerian material is tracked as it flows through the mesh by computing its volume fraction within each element.

Each incremental step in a CEL analysis consists of two phases, Lagrangian and Eulerian. An updated Lagrangian calculation is conducted in an explicit integration scheme, followed by an Eulerian phase in which advection is performed to map the solution variables (such as material properties, stresses, strains, velocities and accelerations) from the deformed mesh to the original mesh. The Eulerian material boundaries and interfaces are updated through the volume fractions of each material and generally do not correspond to element boundaries.

The contact interactions between different Eulerian materials (sand and clay) were not defined because the materials deform continuously, and there is no slip between them. The interaction between the Eulerian materials (soils) and the Lagrangian material (spudcan) was described with a ‘general contact’ algorithm that is based on frictional contact using the penalty method. Rather than the traditional small-sliding frictional formulation, a finite-sliding formulation was implemented to consider arbitrary slide on the interface in large deformation problems. During spudcan penetration into sand overlying clay, contact between the spudcan and sand will be maintained during the entire penetration process. Frictional interaction between the spudcan and sand was defined through a roughness factor (Cassidy and Houlsby, 2002):

\[
\alpha = \frac{\tan \delta}{\tan \phi'}
\]

(4.4)

where \(\delta\) is the interface friction angle between the sand and spudcan.
The current CEL approach in ABAQUS/Explicit has only three-dimensional elements. By taking advantage of geometrical symmetry, only one quarter of the spudcan and soil were modelled. The geostatic stresses due to the submerged unit weight of the soil were imposed before spudcan penetration, with a coefficient of lateral earth pressure of $(1 - \sin \phi')$ for the sand layer and unity for the clay layer.

To enhance solution accuracy, the soil region in contact with the spudcan must be refined. The penetration of a spudcan was simulated with displacement-control. In contrast, spudcan penetration in the field is a load-controlled quasi-static process; hence, the penetration rate adopted in the CEL simulations, which is based on an explicit integration scheme, needs to be sufficiently low to avoid inertial effects. In the penetration resistance profiles presented herein, the depth of penetration, $d$, was zero when the LRP contacts the soil.

### 4.2.2 Constitutive models

The spudcan penetration rates in practical offshore applications are on the order of $(1 - 3)$ m/h (Hossain and Randolph, 2009). Penetration rates in centrifuge model tests are usually selected such that drained deformation occurs in the upper sand layer, while undrained response occurs in the clay layer (Teh et al. 2010; Lee et al. 2013a; Hu et al. 2014). This arrangement is achieved by specifying that the normalised velocity $vD/c_v$ (where $v$ is the absolute penetration rate and $c_v$ is the coefficient of consolidation) is in the range of 30–300 for the clay layer and less than 0.01 for the sand layer (Finnie and Randolph, 1994; Low et al. 2008; Cassidy, 2012).

Loose to medium dense sand usually shows a hardening rather than softening response under drained conditions, so the sand layer was modeled as an elastic-perfectly plastic material with a Mohr-Coulomb yield criterion. The internal friction angle was specified
as the critical value, unless otherwise stated, because (i) the primary aim is to model the
deep penetration resistance in the clay layer following punch-through; (ii) the sand
beneath the advancing spudcan undergoes extremely large deformation and (iii) has
been shown by Li et al. (2013) to reach the critical state during deep penetration (in the
clay layer). The sand volume is stable at the critical state; therefore, the dilation angle
was $\psi = 0$.

The clay layer under undrained conditions was modelled as an elastic-perfectly plastic
material with the Tresca yield criterion. The Poisson’s ratio was 0.49 to approximate
constant volume under undrained conditions. Because soil rigidity only slightly affects
the penetration resistance, a typical Young’s modulus of $500s_u$ was used throughout,
where $s_u$ is the current undrained shear strength of the clay considering softening. The
effect of strain softening is incorporated following Einav and Randolph (2005) by
modifying the shear strengths at the integration points according to the accumulated
absolute plastic shear strain:

$$s_u = \left[ \delta_{rem} + \left( 1 - \delta_{rem} \right) e^{-\xi_{95}} \right] s_u$$

(4.5)

where $s_u$ is the intact undrained shear strength; $\delta_{rem}$ denotes the ratio of fully remoulded
and initial shear strengths (the inverse of the sensitivity, $S_i$); $\xi$ is the accumulated
absolute plastic shear strain; $\xi_{95}$ represents the value of $\xi$ required for the soil to undergo
95% remoulding, estimated in the range of 10 to 50 for marine clays. In CEL
implementations, the plastic shear strain increments in the clay layer were recorded to
update the soil strength at the integration point through Equation 4.5, and the updated
strength was approximated as constant during the calculations of the subsequent step.
The chosen shear strength profile was based on that measured in the centrifuge tests. The intact undrained strength profile was measured using a T-bar penetrometer and was found to increase linearly with soil depth:

\[ s_{ui} = s_{um} + kd \]  

where \( s_{um} \) is the clay strength at mudline for a single clay layer or at the sand-clay interface for sand overlying clay; \( k \) is the strength gradient and \( d \) represents the soil depth. Hossain and Randolph (2009) found that the average shear strain rates induced in soils by spudcan and T-bar penetrometers were comparable; hence, it was unnecessary to consider the potential for shear strength enhancement due to rate effects.

### 4.3 VERIFICATION OF THE CEL APPROACH

#### 4.3.1 Spudcan penetration in a single clay layer

The bearing capacity factor of spudcan in normally consolidated clay has been investigated using different analytical and numerical approaches. A spudcan with diameter of 14 m penetrating in normally consolidated clay was studied by Mehryar and Hu (2002) using the RITSS approach. Detailed spudcan dimensions can be found in Figure 4.1 in Mehryar and Hu (2002). The submerged unit weight of clay, \( \gamma'_c \), was 7 kN/m\(^3\), the undrained shear strength \( s_{ui} = 2d \) kPa and no strain softening was considered. A CEL simulation was performed for comparison, with penetration rate 0.25 m/s, which was sufficiently slow to generate a quasi-static response. The typical element size around the spudcan was 0.036\( D \).
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Figure 4.1. (a) Numerical and analytical analyses for spudcan penetration into NC clay; (b) deformed clay strength profile at penetration depth of 3D

The bearing capacity factors from the CEL, RITSS, traditional small strain FE and plasticity limit analyses are presented in Figure 4.1(a). In the latter two analyses, the spudcan was assumed to be pre-embedded at different depths because the continuous penetration process cannot be tracked. As $d/D > 0.5$, the bearing capacity factor predicted by the CEL is slightly higher than that predicted by the RITSS method, however, the bearing capacity factors from both approaches converge to ~11 at $d/D$ of 3. The bearing capacity factors from both LDFE approaches are moderately lower than that of the wished-in-place spudcans; the bearing capacity is overestimated if the drag-down of weaker soil indicated in Figure 4.1(b) is ignored. The soil radially within ~1D of the centerline of the spudcan is significantly disturbed by the penetration process.

4.3.2 Spudcan penetration in a single sand layer

White et al. (2008) reported a centrifuge test of a conical footing (very similar to a spudcan) with diameter of 4.8 m on medium dense sand ($I_D = 54\%$). A CEL simulation was performed, assuming a Poisson’s ratio of 0.3 and Young’s modulus of 50 MPa. A constant $m$ in correlations by Bolton (1986) was fitted by back-calculation as 0.85 by
White et al. (2008), while $m$ is typically 3 for a triaxial stress state. Corresponding operative friction angles calculated using Bolton’s correlations were 33.22° and 36.32° for these two scenarios, so an average value of 34.8° was assumed in the current analysis, along with a dilation angle of $\psi = 4.8^\circ$ that was similarly based on Bolton’s correlations. A friction coefficient of $\alpha = 0.5$ on the spudcan-sand interface was adopted (the influence of which will be discussed in Section 4.4.2). The penetration rate was 0.1 m/s, and the element size around the spudcan was ~0.03D. The spudcan geometry in the CEL analysis was identical to the model used in the centrifuge tests.

As shown in Figure 4.2, the numerical bearing capacity shows a trend similar to the experimental data with a maximum difference of ~20%. The difference may be due to the constant soil properties assumed in the CEL simulation. During the very early stages of penetration, the friction angle of the sand in reality is likely to be moderately higher than the value used in the CEL analyses because the friction angle in the numerical analyses did not vary with mean stress level. The friction angle may reduce gradually to a nearly constant value with increasing capacity during further penetration, which is consistent with the behaviour modelled in the CEL analysis. For the spudcan penetration in loose or medium dense sand overlying clay, the simple and robust Mohr-Coulomb model was deemed adequate, as the primary concern was capturing the volume, geometry and bearing capacity characteristics in the clay layer of the composite foundation comprising the spudcan and sand plug. The periphery of the sand plug is expected to be subjected to large shear strains during penetration and thus will likely be at the critical state. Therefore, adoption of the Mohr-Coulomb model with a friction angle equal to the critical value is considered appropriate.
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4.4 SIMULATION OF CENTRIFUGE TESTS OF MEDIUM DENSE SAND OVERLYING CLAY

The penetration of a spudcan in medium dense sand overlying clay (see nomenclature in Figure 4.3) behaves differently from that in a single clay or sand layer. CEL simulations were performed to replicate a total of 15 centrifuge tests (relative density $I_D = 43\%$) by Hu et al. (2014). The spudcan model of the centrifuge tests (Figure 4.4), with a $13^\circ$ shallow conical underside profile and $76^\circ$ protruding spigot, was simulated for direct comparison with the experiments. The spudcan diameters and soil properties of each test are listed in Table 4.1. The Poisson’s ratio and Young’s modulus of sand were assumed to be 0.3 and 25 MPa, respectively. The critical state friction angle was $31^\circ$ for the super fine sand used in the centrifuge test (White et al. 2008). The sensitivity of the underlying kaolin clay, $S_t$, was 3, based on cyclic T-bar measurements.

![Figure 4.2. Comparison of CEL with a centrifuge test for spudcan penetration into medium dense sand](image)

**Figure 4.2.** Comparison of CEL with a centrifuge test for spudcan penetration into medium dense sand
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A typical test, L1SP1, with $D$ of 6 m and normalised sand thickness $H_s/D$ of 1, was used as an example to demonstrate the mesh convergence, penetration rate for quasi-static simulation and effects of spudcan roughness and sensitivity of clay on penetration resistance.

![Figure 4.3. Nomenclature for spudcan foundation penetration in sand overlying clay](image_url)

![Figure 4.4. Prototype dimensions of spudcan model (note: the dimensions are absolute values, and the spigot tip-to-shoulder heights vary with $D$)](image_url)
Table 4.1. Prototype parameters and results of centrifuge and numerical tests of medium dense and dense sand overlying clay

<table>
<thead>
<tr>
<th>Test name</th>
<th>Geometry</th>
<th>Sand</th>
<th>Clay</th>
<th>Test</th>
<th>CEL</th>
<th>Prediction method</th>
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<td></td>
<td>Hs (m)</td>
<td>D (m)</td>
<td>Hs/D</td>
<td>l0 (%)</td>
<td>γ'c (kN/m³)</td>
<td>s&lt;sub&gt;sm&lt;/sub&gt; (kPa)</td>
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<td>0.44</td>
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<tr>
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<td>16</td>
<td>0.39</td>
<td>92</td>
<td>10.99</td>
<td>17.7</td>
</tr>
</tbody>
</table>

* Dense sand spudcan test from Lee (2009)
Chapter 4: Predicting the resistance profile of a spudcan penetrating sand overlying clay

4.4.1 Mesh convergence and penetration rate

Mesh convergence studies were conducted to ensure that the Eulerian mesh was sufficiently fine to avoid overestimation of the penetration resistance. Four typical element sizes, indicated as a ratio of the spudcan diameter, were adopted close to the spudcan: $0.018D$, $0.025D$, $0.03D$ and $0.05D$. The meshes had corresponding numbers of soil elements of 490,821, 220,031, 142,560 and 49,197, respectively.

![Finite element mesh used in a typical CEL analysis](image)

**Figure 4.5.** Finite element mesh used in a typical CEL analysis

In order to minimise the computational cost, only a quarter of the domain was modelled. The mesh with element size of $0.025D$ was shown in Figure 4.5. The spudcan was displaced at a penetration rate of 0.2 m/s. The penetration resistance profiles for the different mesh densities are shown in Figure 4.6(a). For an element size of $0.05D$, the
penetration resistance was overestimated significantly. In contrast, load-displacement responses based on element sizes of 0.018D and 0.025D converged, suggesting that mesh convergence is achieved with an element size of 0.025D.

The penetration rate specified to result in a quasi-static analysis must be sufficiently slow, however, the computational cost increases with reduced penetration rate (Tho et al. 2012). The penetration resistance profiles for different penetration rates of 0.1, 0.2, 0.5 and 1 m/s are shown in Figure 4.6(b). The profiles corresponding to the penetration rates of 0.1 and 0.2 m/s show divergence of less than 2%. When the spudcan diameter is larger than 6 m, the inertial effect will become increasingly negligible.

Based on these sensitivity analyses, an element size close to the spudcan of 0.025D and spudcan penetration rate of 0.2 m/s were selected for the remainder of the analyses. These parameters balance computational accuracy and efficiency.

![Figure 4.6. (a) Effect of mesh density (b) Effect of penetration rate](image)

### 4.4.2 Effect of spudcan roughness and $S_t$ on penetration resistances

The effect of spudcan roughness on penetration resistance depends on the soil type. For a rough spudcan in a single clay layer, the penetration resistance is only $\sim 5\%$ higher
than that of a smooth spudcan from CEL simulation (Qiu and Henke 2011), while the difference is calculated to be greater from the $N_c$ expression in Houlsby and Martin (2003). In contrast, the roughness of a spudcan has a distinct influence on the penetration resistance in a single sand layer. Qiu and Henke (2011) also conducted CEL simulations of spudcan penetration on a single sand layer and found that the penetration resistance was enhanced significantly when the friction coefficient $\alpha$ was increased from 0 to 0.5 but only increased by up to a further ~ 5% at deep penetrations for $\alpha > 0.5$.

The numerical penetration resistance profiles for sand overlying clay with $\alpha$ of 0, 0.5 and 1 are shown in Figure 4.7. Overall, the resistance profile with $\alpha$ of 0.5 is close to that with $\alpha$ of 1. The penetration resistance in the clay layer with $\alpha = 0$ is significantly lower than those with $\alpha = 0.5$ and 1, with the divergence becoming increasingly apparent with increasing penetration depth (with a maximum difference of 22% at $d/D = 2.3$). The peak penetration resistance, $q_{\text{peak}}$, which appears in the sand layer, is almost independent of the spudcan roughness. The peak penetration resistance of a smooth spudcan ($\alpha = 0$) appears marginally higher than those of intermediate or fully rough spudcans. The potential cause of this difference is the greater amount of soil flow mobilised higher shear strength in the sand layer, while for the rough spudcan such mobilisation is not obvious and the failure mechanism concentrates mainly at the spudcan-sand interface. When the spudcan is penetrated into the clay layer, stronger sand is trapped beneath the advancing spudcan, and the shape and volume of the sand plug depends on the roughness of the spudcan. Compared with the smooth spudcan, more sand is trapped beneath the frictional spudcan, and the sand plug transfers the loading to deeper soil with higher local undrained strength. A roughness of $\alpha = 0.5$ was used in the following analyses to capture this effect.
Another potential concern in these simulations is that the shaft is treated as frictional like the spudcan to faithfully simulate the experiments of Hu et al. (2014), where the vertical load was measured at the top of the shaft. The impact of this detail was considered by performing an additional analysis for the smallest (6 m diameter) spudcan (for which the impact of friction on the shaft is largest given the same shaft was used for all spudcan sizes). In this analysis, the spudcan was assumed frictional with $\alpha = 0.5$, while the shaft was frictionless. At a penetration depth of $H_s + D$ (a depth relevant to the interpretation of bearing capacity factors in the underlying clay layer presented later), the penetration resistance was reduced by no more than 7% for the frictionless shaft analyses compared to the frictional shaft counterpart. This result indicates that the impact of friction on the shaft was minimal.
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Figure 4.8. Effect of soil sensitivity on penetration resistance profiles

For most offshore soft to medium soft clays, the typical range of soil sensitivity is $S_t$ of 2-5 (Kvalstad et al. 2001; Andersen and Jostad, 2004). The sensitivity of kaolin clay in centrifuge tests was measured by Hu et al. (2014) as $S_t$ of 3, which is close to $S_t$ of 2.5 by Zhang et al. (2011) and $S_t$ of 2-2.5 by Gan et al. (2012) for the same soil. The penetration resistance profiles for $S_t$ of 1, 3 and 5 are shown in Figure 4.8. The profiles with $S_t$ of 3 and 5 are nearly identical, while the penetration resistance is increased by 9% when the sensitivity is reduced from 3 to 1. A sensitivity of 3 is adopted in the following analyses for consistency with the experimentally measured sensitivity.

4.4.3 Full penetration resistance profiles

All 15 centrifuge tests reported by Hu et al. (2014) were grouped with three sand layer thicknesses: $H_s$ of 3.2, 5 and 6 m. Two typical experimental penetration resistances for each sand layer height and corresponding numerical simulations are plotted in Figure 4.9. Reasonably agreement is broadly evident between the CEL results and experiments.
Figure 4.9. Penetration resistance profiles from centrifuge tests and CEL analyses

In general, backflow of the sand on top of the spudcan that occurred in the centrifuge tests was also captured in the CEL simulations. From touchdown of the tip of the spudcan to full embedment into the sand surface, the penetration resistance increases insignificantly, and soil flow is constrained within the sand layer. After the LRP touches the sand, the penetration resistance begins to be rapidly mobilised up to the peak resistance, \( q_{\text{peak}} \), at a penetration depth of 0-2 m in the sand layer. The magnitude of \( q_{\text{peak}} \) depends on both the sand and clay strengths because, at this point, the sand frustum beneath the spudcan is being pushed into the underlying clay layer. The penetration
resistance remains approximately constant or reduces slightly until the LRP of the spudcan penetrates into the underlying clay layer. The punch-through mechanisms observed in the centrifuge tests are generally captured by the simulations, despite the resistances in clay layer being overestimated for some cases. Following penetration of the spudcan and sand plug into the clay layer, the penetration resistance tends to increase proportionally with depth, which is mainly attributed to the linear increase in clay strength with depth. This result suggests that the shape of the composite spudcan and sand plug foundation is essentially stable; thus, the bearing capacity factor may remain constant with depth.

4.4.4 Peak resistance and sand plug height

The peak resistance and corresponding penetration depth must be quantified in routine designs. The peak resistances measured in the centrifuge tests and predicted by the CEL approach and a modified failure stress-dependent model (Hu et al. 2014, and summarised in Section 6 of this paper) are compared in Figure 4.10.

![Figure 4.10. Peak penetration resistances from centrifuge tests, CEL analyses and predicted formulations](image_url)
The predictions from the numerical analyses and analytical model are in reasonable agreement within bounds of ±15%. The CEL analyses generally slightly under-predict the experimental peak resistance due to the assumption in the CEL analyses that the friction angle of sand is equal to the critical value. The analytical model calculations for the same experiments indicate transient operative friction angles slightly higher, 33°-34° at peak resistance. The critical friction angle was used here to appropriately model the shape and volume of the trapped sand plug where the sand at the periphery would be at the critical state (Li et al. 2013).

During penetration in the clay layer, the height of the sand plug, $H_{\text{plug}}$, is important, as it influences the bearing capacity factor and the buoyancy term of Equation 4.1. $H_{\text{plug}}$ is a
function of soil properties, spudcan diameter and test geometry. For the centrifuge experiments described by Hu et al. (2014), all test locations were dissected to examine the deposited height of the trapped sand plug left in the clay after spudcan extraction. Typical sand plug geometries from tests L1SP1 and L1SP4 and the corresponding CEL simulations are presented in Figure 4.11. All the sand plug heights $H_{\text{plug}}$ inferred from the numerical simulations are close to those determined in the post-test measurements illustrated in Figure 4.12(a), while Figure 4.12(b) shows that both the experimental and CEL analyses resulted, on average, in trapped sand plug heights that were 90% of the sand layer thickness. This result is close to the value of $\sim 1H_s$ observed by Teh et al. (2008) and agrees with the upper bound estimated by Lee (2009). The bearing capacity in the clay layer, accounting for buoyancy, can thus be expressed as:

$$q_{\text{clay}} = N_c s_{u0} + H_{\text{plug}} \gamma'_c = N_c s_{u0} + 0.9H_s \gamma'_c \quad \left(0.16 \leq \frac{H_s}{D} \leq 1\right)$$

(4.7)

Figure 4.12. (a) Comparison of CEL numerical and experimental $H_{\text{plug}}$; (b) Relationship between $H_{\text{plug}}$ and $H_s$

4.5 PARAMETRIC STUDIES

4.5.1 Influence of sand thickness and undrained strength of clay
Before predicting the full penetration resistance profile, the influences of several critical factors ($H_s/D$, $s_{um}$ and $k$) were quantified within realistic bounds relative to offshore practices. A complementary set of parametric analyses were thus performed in addition to the experimental simulations, geometric and soil property details of which are listed in Table 4.2. The sand thickness ratio $H_s/D$ was varied from 0.3 to 0.9; clay strength at the sand-clay interface $s_{um}$ from 10 to 40 kPa; clay shear strength gradient $k$ from 1 to 2 kPa/m; and operative friction angle $\phi'$ from 30° to 33° (representative of typical critical state friction angles for siliceous sands). The spudcan geometry (Figure 4.4) was identical to that used in the experiments reported by Hu et al. (2014).

<table>
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<tr>
<th>#</th>
<th>$H_s$ (m)</th>
<th>$D$ (m)</th>
<th>$H_s/D$</th>
<th>$s_{um}$ (kPa)</th>
<th>$k$ (kPa/m)</th>
<th>$\phi'$ (°)</th>
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The normalised sand thickness, $H_s/D$, has an obvious influence on the penetration resistance, as shown in Figure 4.13(a). A punch-through potential is observed at $H_s/D =$
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0.7, with a reduction in penetration resistance predicted during the vertical penetration. An approximately constant peak penetration resistance over the penetration depth is observed in the case of $H_s/D = 0.6$. In the jack-up industry this is often referred to as a rapid-leg-run event. For smaller sand thicknesses, such as $H_s/D = 0.3$, safe installation is possible. This trend is due to the magnitudes of the sources of resistance that comprise $q_{peak}$. For larger $H_s/D$, shearing in the sand layer provides a far larger proportion of $q_{peak}$ than the bearing capacity of the underlying clay layer. In contrast, for small $H_s/D$, the clay bearing capacity is dominant and provides the majority of $q_{peak}$. The clay bearing capacity is mainly dependent upon $s_{sum}$ and $k$, which do not change between these analyses; thus, catastrophic punch-through failure is more likely to occur with a higher $H_s/D$, as the difference between peak and post-peak resistance is greatest. Figure 4.13(a) also shows that, for different sand thickness ratios, the penetration resistances in the clay layer do not converge due to the differing volume and geometry of the trapped sand plugs beneath the spudcan.

![Figure 4.13. Effects of $H_s/D$, $k$ and $s_{sum}$ on penetration resistance profiles](image-url)
The effects of clay shear strength at the sand-clay interface, \( s_{um} \), and strength gradient \( k \) on the penetration resistance are demonstrated in Figure 4.13(b), with \( H_s/D = 0.6 \). Rapid-leg-run potential is observed in all cases, and the depths of peak penetration resistance are nearly independent of the undrained strength of clay; however, the deep penetration resistance in the clay layer increases significantly with increasing \( s_{um} \) or \( k \).

For \( k = 2 \) kPa/m, \( H_s/D = 0.6 \) and \( \phi' = 32^\circ \), the peak resistance for \( s_{um} \) of 20 kPa is a factor of \(~1.3\) times that for \( s_{um} \) of 10 kPa. After the LRP reaches the original sand-clay interface, the gradient of the penetration resistance profiles is nearly independent of \( s_{um} \), while the increasing rate of penetration resistance is larger for higher \( k \) after the spudcan is fully embedded in the clay layer. These trends are further indication that, for a constant geometric ratio \( H_s/D \), a constant \( N_c \) should be used.

### 4.5.2 Equation for bearing capacity factor

When the spudcan and the sand plug penetrate into clay together, the sand plug mobilises soil with higher soil strength than that at the LRP due to the shear strength of the soil increasing with depth. Thus, the bearing capacity factors for spudcan penetration into a single clay layer are inappropriate for the sand overlying clay scenario investigated here.

Bearing capacity factor \( N_c \) was derived by dividing the bearing pressure by the intact undrained shear strength adopted in both the simulations of the centrifuge tests and the complementary parametric analyses. Both \( s_{u0} \) and \( N_c \) refers to the values at LRP in Figure 4.4. When the LRP was penetrated to a depth deeper than \( 1D \) below the sand-clay interface, i.e., \((d-H_s)/D \geq 1\), it is observed that the spudcan with the sand plug underneath has reached a deep ‘steady-state’ according to the CEL simulations of all
centrifuge tests and complementary parametric studies. The capacity factor at 1D below sand-clay interface can be fitted linearly against the sand thickness:

\[ N_{c,\text{deep}} = 15 \frac{H_s}{D} + 9 \quad \left(0.16 \leq \frac{H_s}{D} \leq 1\right) \]  (4.8)

with a coefficient of determination of \(R^2 = 0.93\). This deep factor remains constant with further penetration in clay.

If the capacity factor at depth of \((d-H_s)/D < 1\) is approximated with Equation 4.8 (i.e., \(N_c = N_{c,\text{deep}}\) from the depth of sand-clay interface), the capacity factor is very close to the relationship given in Equation 4.2 that was fitted against the centrifuge tests for very dense sand overlying clay (Lee et al. 2013a). Figure 4.14 demonstrates the normalised \(N_c\) profiles from the sand-clay interface to a depth of 2D below the sand-clay interface. In this figure, the capacity factor near the sand-clay interface \((d = H_s)\) tends to be higher than \(N_{c,\text{deep}}\). This is due to the trapped sand plug not being fully formed, causing some extra sand to shear close to the sand-clay layer interface. However, the typical depth at the end of punch-through events, as demonstrated below, is deeper than the depths affected by the incomplete formation of the sand plug. Hence, for simplicity, \(N_c\) might be set equal to that estimated using Equation 4.8. For the majority of the analyses, the simple linear expression describes the bearing capacity factor with depth to within ±10% of the back-calculated values.
Figure 4.14. Bearing capacity factors from CEL simulations of centrifuge tests and parametric studies

It should be noted that Equation 4.8 was summarised for $H_s/D$ of 0.16 to 1, $\phi'$ of 30 to 33°, $s_{um}$ of 10 to 40 kPa, $k$ of 1 to 2 kPa/m and sensitivity $S_t$ in the range of 3 to 5, all of which represent practical ranges of soil properties for offshore locations, where punch-through failure are potential risks. This equation may not be valid outside of these bounds or for soils with significantly differing behaviours (such as carbonate silts or highly sensitive clays).

4.6 SIMPLIFIED METHOD FOR PREDICTION OF FULL PENETRATION RESISTANCE PROFILE

4.6.1 A simplified full profile prediction method

A simplified method for prediction of a penetration resistance profile for spudcan foundations on sand overlying clay is useful in the evaluation of the potential severity of a punch-through failure. The construction of a simplified resistance profile requires preliminary knowledge of three critical stages in $q \sim d$ space: i) the penetration
resistance from the tip of the spudcan touching the soil until the spudcan spigot (if present) is fully embedded, (ii) the peak resistance in the sand layer, $q_{\text{peak}}$, and (iii) the penetration resistance when the LRP is at the depth of the original sand-clay interface ($d = H_s$) and beyond. At stage (i), the resistance can be assumed to be zero for simplicity because the contribution of a small spigot to the overall capacity is negligible, and the corresponding penetration depth is the height of the spigot. At stage (ii), the modified failure stress-dependent model of Hu et al. (2014) can be utilized:

$$q_{\text{peak}} = \left( N_{c0} \gamma_s \mu_m + q_0 + 0.12 \gamma_s' H_s \right) \left( 1 + \frac{1.76 H_s}{D \tan \psi} \right)^{E^*}$$

$$+ \frac{\gamma_s' D}{2 \tan \psi (E^* + 1)} \left[ 1 - \left( 1 - \frac{1.76 H_s}{D \tan \psi} E^* \tan \psi \right) \left( 1 + \frac{1.76 H_s}{D \tan \psi} \right)^{E^*} \right]$$

where $N_{c0}$ is the bearing capacity factor for clay at the base of a circular foundation, which is obtained using the relationship proposed by Houlsby and Martin (2003) for circular foundations with shear strength increasing linearly with depth, and $q_0$ is the effective overburden pressure at the depth of the foundation. $E^*$ is a parameter to simplify the algebra:

$$E^* = 2 \left[ 1 + D_F \left( \frac{\tan \phi^*}{\tan \psi} - 1 \right) \right]$$

where $D_F$ is a distribution factor that relates the local stress along the failure surface to the average vertical effective stress, or the ratio of the normal effective stress at the slip surface to the mean vertical effective stress, and $\phi^*$ is a reduced friction angle caused by non-associated flow that can be expressed as (Drescher and Detournay, 1993):

$$\tan \phi^* = \frac{\sin \phi \cos \psi}{1 - \sin \phi \sin \psi}$$
The depth of the peak penetration resistance, $d_{peak}$, is 0.12$H_s$, as suggested by Teh et al. (2010) and verified through centrifuge tests for very dense and medium dense sand overlying clay (Hu et al. 2014). For stage (iii) and beyond, the penetration resistance is calculated by substituting $N_{c,\text{deep}}$ in Equation 4.8 for $N_c$ in Equation 4.7. The punch-through distance is thus the depth at which the capacity by Equation 4.7 is equal to $q_{peak}$ minus $d_{peak}$.

### 4.6.2 Performance of the simplified prediction method

To explore the performance of the proposed simplified prediction method, both the resistance profile and the depths of punch-through event were compared for 20 centrifuge tests of sand overlying clay in Table 4.1 (15 for medium dense sand overlying clay from Hu et al. (2014) and 5 for very dense sand overlying clay from Lee et al. (2013a)). Figure 4.15 demonstrates the comparisons for two pairs of typical tests from medium dense sand overlying clay and very dense sand overlying clay. In general, the initial sharp increase in the resistance on sand and the resistance in the clay layer are predicted reasonably well for the four cases. This validates the model of Hu et al. (2014) for $q_{peak}$ prediction and the equations proposed here for $q_{clay}$ (Equation 4.7 and Equation 4.8). If the depth of the peak resistance is also known, which has been demonstrated to be readily predicted (Figure 3.6 and Equation 3.3), the potential maximum depth of a punch-through event from onset until equilibrium is re-established can be estimated. It is acknowledged that for jack-up units that can perform preloading at draft this maximum distance may not be the primary concern, and the shape of the resistance profile after peak may be more critical. Though this is not provided in this simplified approach, the $d_{\text{punch, predicted}}$ value here gives an indication of severity.
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Figure 4.15. Nomenclature of $d_{\text{punch, measured}}$ and $d_{\text{punch, predicted}}$ for (a) medium dense and (b) very dense sand tests

This figure also demonstrates that, even though it was derived from CEL simulations for loose and medium dense sand overlying clay, the simple prediction model for bearing capacity in the clay layer still appears valid for very dense sand overlying clay scenarios. The soil at the periphery of the sand plug have reached the critical state during penetration in the clay layer – irrespective of the initial relative density of the sand layer – due to the cumulative shearing to which it is subjected. Hence, simulations setting the operative friction angle equal to the critical value also appear valid for very dense sand overlying clay. It would be beneficial in the future to simulate the dense to very dense sand overlying clay centrifuge tests using a constitutive model for the sand that can account for strain hardening and softening.

The predictions of the simplified method for both medium dense and very dense sand tests are also presented in terms of $d_{\text{punch, predicted}}/d_{\text{punch, measured}}$ in Figure 4.16. Though the predictions from the simplified method underestimate the uncontrolled penetration depth, the under-prediction is less than 20% for the majority of the tests. Even with an under-prediction of 20%, the method clearly allows a relatively fast and accurate
prediction of the punch-through distance of a punch-through event that can be compared against the operability limits of the jack-up rig. All parameters are routinely derived during site investigation campaigns prior to jack-up rig deployment. It should be noted that the framework presented is verified by centrifuge tests of medium dense and very dense sand overlying kaolin clay; more testing and field data with a wider range of material properties would further validate its performance.

Figure 4.16. Performance of the simplified prediction method by comparison of rapid penetration depth with that obtained from centrifuge tests

4.7 CONCLUSIONS

This paper reports the results of CEL simulations of centrifuge tests, investigating the potential for punch-through of spudcan foundations during deep penetration through a sand layer into an underlying clay layer. Generally reasonable agreement was obtained between the results of centrifuge tests and CEL analyses in terms of penetration resistance profiles. A simplified prediction method with straightforward algebraic
expressions and clearly defined parameters for producing a complete simplified penetration resistance profile for foundations on sand overlying clay has been presented.

By incorporating strain softening clay constitutive model in the simulation, both punch-through and rapid-leg-run potential in medium dense sand overlying clay centrifuge tests were replicated. The spudcan-soil contact property was shown to have only a small effect on $q_{\text{peak}}$, though more significant differences in $q_{\text{clay}}$ were found for smooth and frictional cases in which the spudcan was fully embedded into the clay layer. The sensitivity of the clay, $S_t$, had a small effect on the penetration resistance when within the range of $3 \leq S_t \leq 5$.

Using complementary parametric analyses, the effects of $H_s/D$, $k$ and $s_{\text{um}}$ on the penetration resistance profile were investigated. $H_s/D$ mainly controls the pattern of failure potential, with larger values being more prone to punch-through failure (reduction of vertical load with depth), while rapid-leg-run (approximately constant peak load with depth) is more likely for smaller values of $H_s/D$. When $H_s/D$ was further decreased, a non-linear increase of the penetration resistance was apparent, indicating no punch-through or rapid-leg-run potential. The deep penetration resistance, $q_{\text{clay}}$, increased significantly with increasing $s_{\text{um}}$, while the gradient of $q_{\text{clay}}$ was largely independent of $s_{\text{um}}$ for clay with the same $k$. A linear expression for the bearing capacity factor with respect to $H_s/D$ for the spudcan and underlying plug is summarised from numerical simulations of centrifuge tests and complementary analyses. The equation is based on the geometric conditions and material properties relevant to spudcan punch-through and should be used with caution for any cases beyond the conditions explored in this manuscript (such as non-siliceous sands, highly sensitive clays or clay with $s_{\text{um}} > 40$ kPa).
In the simplified prediction method, for penetration resistance in the upper sand layer, the stress level and dilatant response as well as the embedment depth are taken into account; for penetration in the underlying clay, the proposed design equations incorporate the shear strength increment with depth and the thickness of the trapped sand beneath the foundation. In retrospectively calculating the penetration resistance profiles for all 15 medium dense sand and 5 very dense sand centrifuge tests, the predicted full penetration resistance profiles show generally good agreement with the testing profiles. The simplified prediction method under-predicts the rapid penetration depth by less than 20%.

4.8 REFERENCES


Chapter 4: Predicting the resistance profile of a spudcan penetrating sand overlying clay


CHAPTER 5. THE EFFECT OF CONICAL SHAPE ON A FOOTING PENETRATING SAND OVERLYING CLAY

ABSTRACT
This paper reports a series of centrifuge model tests investigating the effect of shape on the penetration resistance of spudcan and conical footings on sand overlying clay. The effect of footing shape and geometry on single layer soil has been studied intensively. However, there is still limited understanding of how the shape of conical footings affect their response on sand over clay. This has been addressed in this study by observing experimentally the punch-through failure mode induced during the penetration of a footing through sand into underlying clay. Digital images were captured in a geotechnical centrifuge during penetration of various shapes of half-footing held against a transparent window of a strongbox. The images were then analysed using particle image velocimetry techniques coupled with close-range photogrammetric correction of the measurements. The full penetration resistance profiles and accompanying mechanisms of deformation for footings with different conical angles on sand overlying clay are presented. The experimental evidence shows that, irrespective of the conical angle (angle of the base conical section) within the range of 7° to 21° (the range used for the spudcan footings of offshore mobile platforms), when the footing penetrates through sand into an underlying clay layer: (i) accumulated radial and deviatoric shear strains along the future failure surface counteract each other, resulting in similar peak resistance in the sand layer, and (ii) a trapped sand plug of constant height is pushed into the underlying clay layer. These observations serve to justify previously proposed methods for predicting (i) the peak resistance of a conical footing on sand overlying clay and (ii) the bearing capacity profile for a footing and underlying sand plug.
penetrating through a deep clay layer, both of which are required to predict the potential for, and severity of, punch-through type failure.

5.1 INTRODUCTION

Mobile jack-up units are widely used for offshore drilling in water depths up to ~150 m. Modern jack-up rigs are typically supported by three independent truss legs, with each attached to a large inverted conical footing called a spudcan. Spudcan designs change dependent on the manufacturer, with conical angles usually ranging from 0° to 20°, though some can be even sharper up to 40° (Menzies and Roper 2008; Dean 2010). When considering the installation of jack-up foundations, it is necessary to predict their bearing capacity with depth to assess the potential for ‘punch-through’ type failure. Punch-through occurs when the bearing pressure suddenly reduces following failure of a stiff top layer overlying a softer layer, such as sand overlying clay. This type of failure has been modelled extensively in the centrifuge (Craig and Chua 1990; Teh et al. 2008, 2010; Lee et al. 2013a; Hu et al. 2014a). Prediction methods for estimating ‘punch-through’ have been developed based upon these experiments (Lee et al. 2013b; Hu et al. 2014a; 2014b) by estimating the peak resistance in the sand layer, $q_{\text{peak}}$, and the depth of the equivalent resistance in the underlying clay layer, $q_{\text{clay}}$. These are potentially useful – once validated – in helping jack-up operators to quantify the risk of ‘punch-through’ based on the spudcan geometry and seabed conditions (Osborne et al. 2009). However, the sensitivity of footing geometry on the governing deformation mechanisms during penetration of a footing through sand overlying clay has not been sufficiently investigated for such methods to be used for different spudcan shapes with confidence. The primary focus of this paper is to address this shortcoming experimentally and assess the validity of the previously proposed approaches for a range of spudcan geometries.
The peak penetration resistance, $q_{\text{peak}}$, in sand layer and the resistance in clay layer, $q_{\text{clay}}$, are two critical components in approximating the full penetration resistance profile analytically. Through a series of spudcan (for an equivalent conical angle of 13°) and flat footing penetration tests on very dense sand (relative density $I_D = 92\%$) overlying clay, a failure stress dependent model was proposed by Lee et al. (2013b) to calculate $q_{\text{peak}}$. This model assumes an inverted and truncated cone of sand with inclination angle equal to the dilation angle is pushed down into the underlying clay. Hence the value of $q_{\text{peak}}$ consists of the frictional resistance from the shearing sand and the bearing resistance from the underlying clay.

![Diagram](image)

**Figure 5.1. Modified failure-stress-dependent mechanism accounting for embedment depth attained during mobilisation of $q_{\text{peak}}$ (after Hu et al. 2014a)**

The model was modified by Hu et al. (2014a) (see Figure 5.1) to account for mobilisation embedment depth and validated experimentally for medium dense to very dense sand, again for a spudcan geometry with main conical angle of 13°, resulting in the following expression:
Chapter 5: The effect of conical shape on a footing penetrating sand overlying clay

\[ q_{\text{peak}} = \left( N_{c0} s_{um} + q_0 + 0.12 \gamma' s H_s \right) \left( 1 + \frac{1.76 H_s}{D} \tan \psi \right)^E \]

\[ + \frac{\gamma' s D}{2 \tan \psi (E^s + 1)} \left[ 1 \left( 1 - \frac{1.76 H_s}{D} \tan \psi \right) \right] \left( 1 + \frac{1.76 H_s}{D} \tan \psi \right)^E \]

where \( N_{c0} \) is the bearing capacity factor for clay at the base of a circular foundation, which is obtained using the lower bound solutions proposed by Houlsby and Martin (2003), \( s_{um} \) is the undrained shear strength of clay at sand-clay interface; \( q_0 \) is the effective overburden pressure at the depth of the foundation; \( \gamma' \) is the effective unit weight of sand; \( H_s \) is the initial sand layer thickness; \( D \) is the spudcan diameter; \( \psi \) is the dilation angle of sand. Lastly, \( E^s \) is a parameter to simplify the algebra:

\[ E^s = 2 \left[ 1 + D_F \left( \frac{\tan \phi^*}{\tan \psi} - 1 \right) \right] \]

where \( D_F \) is a distribution factor and \( \phi^* \) is a reduced friction angle caused by non-associated flow rule that can be expressed as (Drescher and Detournay, 1993):

\[ \tan \phi^* = \frac{\sin \phi \cos \psi}{1 - \sin \phi \sin \psi} \]

The expression for \( q_{\text{peak}} \) was derived from the integration of the vertical force equilibrium equation on an infinitesimal element, in which the distribution factor \( D_F \) is an empirical factor intended to relate the normal effective stress on the slip surface to the average vertical effective stress on the infinitesimal element in the sand layer beneath the footing. The distribution factor is the only empirical parameter in these models and was found to depend only on the sand thickness ratio \( H_s/D \), which was expressed originally via a pair of linear equations for spudcan (13° conical underside) and flat footings respectively (Lee et al. 2013b). Accounting for mobilisation depth and further experimental evidence after Hu et al. (2014a), the distribution factor was modified, and for spuds cans was recommended as:
Chapter 5: The effect of conical shape on a footing penetrating sand overlying clay

\[
D_f = 0.642 \left( \frac{H_s}{D} \right)^{-0.576} \quad \text{as} \quad 0.16 \leq \frac{H_s}{D} \leq 1.0
\]  
(5.4)

By assuming the same modified mechanism and based on the very dense sand tests of Lee et al. (2013a), a similar power relationship for a flat footing can be found as:

\[
D_f = 0.623 \left( \frac{H_s}{D} \right)^{-0.174} \quad \text{as} \quad 0.21 \leq \frac{H_s}{D} \leq 1.12
\]  
(5.5)

These power relations give a closer fit for a larger range of geometries and material properties than the linear fits of Lee et al. (2013b). The inequalities presented alongside the \( D_f \) relations represent the geometric range of sand layer thickness to spudcan diameter over which the fits were calculated.

When the spudcan penetrates through the sand into the clay, a sand plug is trapped underneath the spudcan and Hu et al. (2014b) showed that the penetration resistance in the clay layer could be adequately expressed as:

\[
q_{\text{clay}} = N_c s_{u0} + H_{\text{plug}} \gamma'_c = N_c s_{u0} + 0.9 H_s \gamma'_c \quad \left( 0.16 \leq \frac{H_s}{D} \leq 1 \right)
\]  
(5.6)

where \( N_c \) is the bearing capacity factor in the clay layer; \( s_{u0} \) is the soil shear strength at the load reference point (taken as the lowest level of the maximum cross-section of the footing); \( H_{\text{plug}} \) is the sand plug height; and \( \gamma'_c \) is the effective unit weight of clay. The first term is the bearing capacity for a composite foundation of spudcan and trapped sand plug in weightless soil, while the second term accounts for buoyancy. Craig and Chua (1990) observed that the trapped sand plug had height equal to \( H_s \) through examination of cross sections of spent centrifuge samples. By conducting particle image velocimetry (PIV) analysis on a half-spudcan model, Teh et al. (2008) also reported the trapped sand plug height was \(~1H_s\) and the upper portion of the trapped sand resembled a cylinder, while the lower part was somewhat irregularly shaped. A value of \( 0.9H_s \) was
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recommended in Teh (2007) for spudcan with 13° underside. Based on centrifuge tests of medium dense sand ($I_D = 43\%$) overlying clay and complementary parametric studies on loose and medium dense sand, $N_c$ was back-calculated by Hu et al. (2014b) and normalised by the deep bearing capacity factor, $N_c\text{, deep}$ (taken as the value of $N_c$ at $1D$ depth below the sand-clay interface). By incorporating the combined geometry of the spudcan and sand plug, an equation for $N_c$ was proposed based on the best fit of $N_c\text{, deep}$:

$$N_c = 15 \frac{H}{D} + 9 \left( 0.16 \leq \frac{H}{D} \leq 1 \right)$$

(5.7)

These design methodologies were all formulated based upon centrifuge tests performed using a spudcan footing of 13° effective conical shape. However, spudcan footings used on jack-up rigs do not always have the same geometry as the conical angle of the underside often varies, with some example shapes used on offshore rigs illustrated in Cassidy et al. (2009) and Dean (2010). The aims of this paper are to investigate the impact of varying the conical angle of the underside of a footing on the validity of Equations 5.1-5.7 by:

1. Modelling the effect of angle of a conical footing on the peak resistance in medium dense sand overlying clay in a centrifuge.

2. Observing the deformation mechanisms during all stages of punch-through using PIV techniques, revealing the effect of footing shape on: a. the peak resistance; b. the trapped sand plug height; and c. the bearing capacity mechanisms in the underlying clay layer.

3. Assessing the performance of the models given by Equations 5.1-5.7 for estimating the ‘punch-through’ risk.
5.2 EXPERIMENTAL METHODOLOGY

A series of half-footing penetration tests were performed in the drum centrifuge at the University of Western Australia (UWA) with the soil flow mechanisms around the footings being observed using the PIV apparatus developed by Stanier and White (2013). Images recorded during the tests were then analysed to reveal deformation mechanisms using geoPIV (White et al. 2003). The testing programme was designed to span a large range of normalised sand thickness, $H_s/D$, thus encompassing the range of practical interest to jack-up operators. The angles of the conical footing models were in the range commonly found offshore of 0°–21°.

5.2.1 Experimental set-up

The experimental setup consisted of a machine vision camera with an image resolution of ~5 million pixels mounted in front of the viewing window and aligned so as to minimise the non-coplanarity of the transparent window and camera. The soil samples were prepared in strongboxes with internal dimensions of 258 mm length × 160 mm width × 160 mm depth, with a transparent sidewall providing an exposed cross section viewing area of 258 × 160 mm. The strongbox was fitted into the ring channel of the drum centrifuge with its exposed plane parallel to the centre of centrifuge rotation. A pair of light emitting diode panels with diffusing filters, positioned along the top and bottom of the strongbox, uniformly illuminated the exposed face of the samples.

Five half-footing models (Figure 5.2), including four simple conical footings and one with an additional spigot, were tested and are referred to as models C0, C7, C14, C21 and S13 respectively. Prefix ‘C’ refers to the conical shaped footings and ‘S’ refers to the model with the additional spigot. The accompanying number refers to the angle of the base conical section. All footings shared a similar upper conical angle of 13°. For
straightforward comparison to previously published data, model S13 had the same geometry as that used by Teh et al. (2008, 2010) and Lee et al. (2013a) and Hu et al. (2014a). All models were semi-circular in shape, with diameter $D$ of 40 mm. The diameter of the footing was chosen so as to be sufficiently large to allow reasonably detailed images of the soil flow patterns to be captured whilst avoiding boundary effects from the edge of the strongbox. To achieve proper sealing and prevent soil ingress between the window and back plate of the footing model, closed-cell foam was attached to the flat surface of the footing and lightly greased before being pressed against the inner side of the transparent window by the actuator. The footing load was measured using an S-shaped load cell; oriented so as to be minimally affected by bending moments generated within the shaft of the spudcan. Buoyancy and friction between the footing and window were measured in repeat tests in strongboxes containing only water. These effects were then subtracted from the response measured for the models containing soil during the analysis process.

![Diagram](image_url)

**Figure 5.2. Dimensions of model footings used in centrifuge test (all measurements in mm)**
5.2.2 Sample preparation

Commercially available super fine silica sand and kaolin clay were used as the upper sand and lower clay layer respectively. Both have been used extensively in previous centrifuge model studies at UWA, with their behaviour well documented (Cheong 2002; Acosta-Martinez and Gourvenec 2006). The material properties of the sand and clay are provided in Table 5.1.

<table>
<thead>
<tr>
<th>Soil</th>
<th>Property</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Super fine silica sand</td>
<td>Specific gravity</td>
<td>( G_s )</td>
<td>2.65</td>
</tr>
<tr>
<td></td>
<td>Average effective particle size</td>
<td>( d_{50} )</td>
<td>0.19 mm</td>
</tr>
<tr>
<td></td>
<td>Maximum void ratio</td>
<td>( e_{\text{max}} )</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>Minimum void ratio</td>
<td>( e_{\text{min}} )</td>
<td>0.45</td>
</tr>
<tr>
<td></td>
<td>Critical state friction angle</td>
<td>( \phi_v )</td>
<td>31°</td>
</tr>
<tr>
<td>UWA kaolin clay</td>
<td>Liquid limit</td>
<td>LL</td>
<td>0.61</td>
</tr>
<tr>
<td></td>
<td>Plastic limit</td>
<td>PL</td>
<td>0.27</td>
</tr>
<tr>
<td></td>
<td>Specific gravity</td>
<td>( G_s )</td>
<td>2.6</td>
</tr>
<tr>
<td></td>
<td>Angle of internal friction</td>
<td>( \phi' )</td>
<td>23°</td>
</tr>
<tr>
<td></td>
<td>Coefficient of consolidation</td>
<td>( c_v )</td>
<td>(~2 \text{m}^2/\text{year})</td>
</tr>
</tbody>
</table>

A bed of clay was formed in each strongbox by pouring kaolin slurry (mixed at 120% water content under vacuum) in-flight at 40g followed by consolidation at 300g. With the clay layer pre-consolidated, dry superfine silica sand was air-pluviated into each chamber at 1g using an electrically controlled bar-type sand hopper. The densities of the sand layers formed were determined from measurements of the total added sand weight and the volume the layer occupied in the strongbox. The average relative density, \( I_D \), was 74% while the standard deviation was 7%, indicating excellent uniformity between strongboxes. This gives the sand layer an effective unit weight \( \gamma' \), of 10.6 kN/m\(^3\) on average. The dry sand bed was partially saturated prior to replacement of the sample in the drum centrifuge channel to provide sufficient capillarity in the sand layer so that the sample did not collapse during remounting.
The final preparation stage prior to testing was to ensure optimal precision of the PIV measurements by applying artificial seeding to the exposed plane of each model. Black and green colored artificial seeding was evenly sprinkled onto the exposed sand and clay layers respectively; enhancing the visual texture of the sand and clay and allowing easy detection of movement. The optimal densities of seeding on the sand and clay layers were defined by following the procedure proposed by Stanier and White (2013). The transparent window was then reinstalled before the strongbox was carefully slotted into the centrifuge channel. The sand layers were back saturated with water during ramping-up and the sample was then subjected to an acceleration level of 300g and allowed to further consolidate. All tests were carried out at an acceleration of 200g; hence the over-consolidation ratio was at least 1.5. This resulted in a gradient of the shear strength within the clay layer that allowed the full punch-through process to be modelled within the confines of the strongbox.

5.2.3 Testing procedure

Once the soil sample was fully consolidated, the penetration test commenced and images were recorded simultaneously. The footing penetration location was positioned close to the centre of the strongbox window, giving a clearance of at least 2D between the footing edge and the sidewall of the strongbox. This clearance was deemed sufficient in view of the fact that the majority of soil flow induced by the spudcan penetration in sand overlying clay occurs within a distance of approximately 1D from the spudcan edge (see Teh et al. 2008). The footing penetration process was displacement-controlled at a rate of $v = 0.19$ mm/s. This displacement rate ensured that the dimensionless velocity, $V = vD/c_v$, was less than 0.01 for the sand layer and approximately 120 in the clay layer, ensuring a drained sand response and undrained clay response (Chung et al. 2006; Cassidy 2012) where $c_v$ is the coefficient of
consolidation. Images were captured continuously in-flight at 5 frames per second, which equates to an image for every 0.038 mm of (model scale) displacement, providing excellent resolution for visualisation of the failure mechanism. The soil displacement trajectories were computed from the images recorded using the PIV technique (e.g., White et al. 2003). The analysis process included application of close-range photogrammetry corrections to convert the image space coordinates into object space coordinates, accounting for image distortions.

To take full advantage of the testing area, two tests were conducted on opposite sides of each strongbox. Two different sand thicknesses were prepared within each soil sample. The thicker sand layer was tested first, and then the channel was brought to a standstill so that sand could be scraped from the surface until the desired sand thickness was achieved ready for the next test. Actual sand layer thicknesses were measured using the same close range photogrammetry techniques for correcting the displacement measurements. On each occasion, the sample was left to reconsolidate with the new sand thickness before commencement of the penetration test. Table 5.2 provides a summary of the 11 tests performed, alongside the measured sand layer heights.

## 5.3 PENETRATION RESISTANCE PROFILES

After correction to account for buoyancy and interface friction between the footing and the transparent window, the remaining force was divided by the cross-sectional area at the load reference point to obtain the penetration resistance, $q$. The undrained shear strength profile of the underlying clay was estimated using miniature T-bar tests conducted after removal of the upper sand layer, assuming an intermediate roughness $N_{T-bar}$ factor of 10.5 (Stewart and Randolph 1991). This avoided the problem of sand becoming trapped beneath the T-bar during penetration through the sand layer, which
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causes an overestimation of shear strength of the clay. Strength profiles were linear with increasing strength with depth, with the strength at the sand-clay interface and strength gradients for all samples summarised in Table 5.2.

<table>
<thead>
<tr>
<th>Test identifier</th>
<th>Hs (m)</th>
<th>D (m)</th>
<th>Hs/D</th>
<th>conical angle (°)</th>
<th>lD (%)</th>
<th>ssum (kPa)</th>
<th>k (kPa/m)</th>
<th>DF</th>
<th>qpeak (kPa)</th>
<th>φ' (°)</th>
<th>Ψ (°)</th>
<th>Nc,dep</th>
</tr>
</thead>
<tbody>
<tr>
<td>H7C7</td>
<td>6.89</td>
<td>8</td>
<td>0.86</td>
<td>7</td>
<td>74</td>
<td>21.86</td>
<td>2.09</td>
<td>0.73</td>
<td>702.82</td>
<td>35.2</td>
<td>5.3</td>
<td>20.97</td>
</tr>
<tr>
<td>H7C14</td>
<td>7.11</td>
<td>8</td>
<td>0.89</td>
<td>13.65</td>
<td>74</td>
<td>22.24</td>
<td>2.11</td>
<td>0.75</td>
<td>758.95</td>
<td>35.1</td>
<td>5.2</td>
<td>–</td>
</tr>
<tr>
<td>H7C21</td>
<td>7.25</td>
<td>8</td>
<td>0.91</td>
<td>21</td>
<td>74</td>
<td>22.22</td>
<td>2.09</td>
<td>0.71</td>
<td>740.25</td>
<td>35.1</td>
<td>5.1</td>
<td>–</td>
</tr>
<tr>
<td>H5C0</td>
<td>4.91</td>
<td>8</td>
<td>0.61</td>
<td>0</td>
<td>74</td>
<td>19.29</td>
<td>2.08</td>
<td>0.61</td>
<td>368.11</td>
<td>36.4</td>
<td>6.7</td>
<td>15.51</td>
</tr>
<tr>
<td>H5C7</td>
<td>5.09</td>
<td>8</td>
<td>0.64</td>
<td>7</td>
<td>74</td>
<td>16.66</td>
<td>1.8</td>
<td>0.83</td>
<td>436.74</td>
<td>36</td>
<td>6.3</td>
<td>16.48</td>
</tr>
<tr>
<td>H5S13</td>
<td>5.13</td>
<td>8</td>
<td>0.64</td>
<td>13</td>
<td>74</td>
<td>19.58</td>
<td>2.08</td>
<td>0.81</td>
<td>487.82</td>
<td>35.8</td>
<td>6</td>
<td>–</td>
</tr>
<tr>
<td>H5C14</td>
<td>5.41</td>
<td>8</td>
<td>0.68</td>
<td>13.65</td>
<td>74</td>
<td>20.72</td>
<td>2.13</td>
<td>0.75</td>
<td>504.69</td>
<td>35.6</td>
<td>5.8</td>
<td>–</td>
</tr>
<tr>
<td>H5C21</td>
<td>5.25</td>
<td>8</td>
<td>0.66</td>
<td>21</td>
<td>74</td>
<td>20.55</td>
<td>2.13</td>
<td>0.72</td>
<td>456.93</td>
<td>35.7</td>
<td>5.8</td>
<td>15.27</td>
</tr>
<tr>
<td>H3C7</td>
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<td>8</td>
<td>0.38</td>
<td>7</td>
<td>74</td>
<td>11.34</td>
<td>1.51</td>
<td>1.2</td>
<td>246.36</td>
<td>37.3</td>
<td>7.8</td>
<td>14.2</td>
</tr>
<tr>
<td>H3C14</td>
<td>3.03</td>
<td>8</td>
<td>0.38</td>
<td>13.65</td>
<td>74</td>
<td>11.31</td>
<td>1.51</td>
<td>1.16</td>
<td>237.28</td>
<td>37.3</td>
<td>7.9</td>
<td>16.65</td>
</tr>
<tr>
<td>H3C21</td>
<td>3.18</td>
<td>8</td>
<td>0.4</td>
<td>21</td>
<td>74</td>
<td>13.93</td>
<td>1.83</td>
<td>0.99</td>
<td>261.95</td>
<td>36.9</td>
<td>7.4</td>
<td>15.1</td>
</tr>
</tbody>
</table>

Table 5.2. Summary of half-footing penetration tests and results
Figure 5.3. Full penetration resistance profiles from PIV tests

The response measured for all tests with similar $H_s/D$ ratios are presented in Figure 5.3 to a final penetration depth of $1D$ from the base of the strongbox, beyond which boundary effects were observed (as modelled numerically by Ullah and Hu 2012). For tests H7C21, H7C14, H5C21 and H5S13, the penetration resistance in the clay layer for depths beyond $1D$ from the sand-clay interface appeared odd due to sudden reductions in the magnitudes of penetration resistances. Assessment of the PIV images showed this was due to an apparent loss of sand plug volume that may have been caused by the footing becoming misaligned with the transparent window at such large depths, thus the responses beyond these depths were ignored.

In general, the penetration resistance began to increase when the tip of the conical footing penetrated into the sand and increased rapidly due to increasing contact area up
until the $q_{\text{peak}}$ was mobilised. Subsequent penetration led to a nearly constant or small reduction in resistance with the minimum penetration resistance observed close to the original depth of the sand-clay interface. Beyond this point the penetration resistance increased gradually with penetration depth due to the increasing clay strength with depth. The measured penetration resistance profiles in terms of footings with different shapes indicate:

- The $q_{\text{peak}}$ for the flat footing (C0) was always lower than tests performed with a conical shaped footing (C7, C14, C21) for a similar $H_s/D$ ratio. This has implications on the form of design equations for predicting $q_{\text{peak}}$ as will be demonstrated later.

- The normalised sand thickness, $H_s/D$, plays a critical role in the magnitude of $q_{\text{peak}}$ mobilised; larger $H_s/D$ leads to higher $q_{\text{peak}}$. This is consistent with the findings published in the literature (e.g., Meyerhof 1974; and for spudcans by Teh et al. 2010; Lee et al. 2013a; Hu et al. 2014a).

- The majority of the tests exhibited near-constant penetration resistance following $q_{\text{peak}}$ before gradually increasing penetration resistance with depth. Potential prototype depths of penetration before increasing capacity were $\sim10$ m; indicating a significant punch-through risk.

## 5.4 PEAK RESISTANCE IN SAND

### 5.4.1 Distribution factors for varying spudcan geometries

Assuming the failure stress dependent failure mechanism of Hu et al. (2014a), the values of $D_f$ for all the penetration tests conducted were back-calculated and are plotted in Figure 5.4 alongside back-calculated values for other recent centrifuge tests on fully circular model spudcans (Lee et al. 2013b; Hu et al. 2014a).
Chapter 5: The effect of conical shape on a footing penetrating sand overlying clay

Figure 5.4. Distribution factors from half-footing PIV tests and previous centrifuge tests (note: solid represents this study and non-solid represents previous experiments of Hu et al. 2014a for shape S13 and Lee et al. 2013b for shape C0)

Lee et al. (2013a) observed that a spudcan has a higher $D_F$ values than an equivalent flat footing for the same $H_s/D$ ratio and postulated that the reason is that higher radial stress and thus resistance along the assumed slip surface in the sand are invoked by the conical underside of the spudcan. Extrapolating from this, it might also be plausible to assume that larger conical angles lead to larger $D_F$ values. However, Figure 5.4 shows this not to be the case since all of the footing tests with conical angles between 7° and 21° exhibit similar $D_F$ values, irrespective of the angle of the conical underside.

Figure 5.5 reveals the cause by comparing the accumulated radial and deviatoric shear strains generated by penetration of two footings with differing conical angles up to the point at which peak resistance was mobilised. The soil displacements were measured using GeoPIV, taking a patch size of 50 pixels and spacing of 10 pixels. The reference image was updated periodically to preserve the correlation in the PIV computations,
allowing large deformations to be tracked. The strains were then obtained from the displacement fields using a finite strain formulation after White and Bolton (2004). The radial strain was calculated as:

\[ \varepsilon_r = \frac{\partial u}{\partial r} \]  

(5.8)

where \( u \) is the radial displacement and \( r \) is the radius. While the deviatoric shear strain was taken as:

\[ \varepsilon_{dev} = \sqrt{(\varepsilon_r - \varepsilon_z)^2 + (\varepsilon_r - \varepsilon_\theta)^2 + (\varepsilon_z - \varepsilon_\theta)^2 + 0.5\gamma_{rz}^2} \]  

(5.9)

where \( \varepsilon_z \) is the normal stain, \( \varepsilon_\theta \) is circumferential strain; and \( \gamma_{rz} \) is the engineering shear strain. The larger conical angle clearly causes greater radial strain in the vicinity of the failure surface in the sand layer, which emanates from the outer radius of the footing. However, the increasing conical angle also causes significantly larger deviatoric shear strains to accumulate within the same region, which causes strain softening of the sand reducing its mobilised friction angle.

Figure 5.5. Accumulated (a) radial and (b) deviatoric shear strain distribution (%) for test H3C7 and H3C21

Figure 5.6 plots the average accumulated radial and deviatoric shear strains measured within 0.05\( D \) of the inclined failure surface in the sand layer assumed in the failure
stress dependent model of Hu et al. (2014a) versus the conical angle of the underside of the footings. Strain measurements were averaged within $0.05D$ of the failure surface to provide an estimate of the average strain that occur within the shear band formed at the assumed failure surface. An averaging distance of $0.05D$ implies a shear band thickness of $0.05D$ or 10 times the $d_{50}$ of the sand particles, which is the size of a ‘homogeneously heterogeneous’ shear zone according to Muir Wood (2012). The average is taken so as to provide an overall estimation of the impact of the embedment of the conical underside of the footing. The range in the accumulated strains (both radial and deviatoric shear) for a given conical angle in Figure 5.6 is a result of the range of $H/D$ ratios tested (see Table 5.2), which influences the calculation of the average accumulated strains.

![Figure 5.6. Average accumulated radial and deviatoric shear strains along the shear band of the soil underneath the footing](image)

There is a clear linearly increasing trend for both strains with increasing conical angle. Thus, the increase of $D_F$ due to higher stress is compensated for, at least partially, by the
softening caused by the accumulated deviatoric shear strains. This perhaps explains why there is no systematic difference in $D_F$ for footings with conical angles between $7^\circ$ and $21^\circ$. Since the majority of the footings used in the field fall within this range (Menzies and Roper 2008), the power relationship for $D_F$ for spudcan footings given by Equation 5.4 is suitable for use in estimating $q_{peak}$ for spudcans with geometries within the bounds investigated here.

Figure 5.7. Back-calculated friction and dilation angles mobilised at peak resistance (line fit represents modified Bolton’s equations proposed by Lee et al. 2013b and Hu et al. 2014a, as provided in insert)

Although the sand samples prepared in the centrifuge tests had similar relative density ($\sim 74\%$), the ‘average’ friction and dilation angles mobilised at peak resistance depends on the footing sizes and sand thickness. The modified failure stress dependent model of Hu et al. (2014a), which is based upon the stress-dilatancy relationships of Bolton (1986), has the ability to reflect the stress-level dependency at peak resistance. The mobilised friction and dilation angles corresponding to peak resistance for the tests in Table 5.2 were back-calculated and are shown in Figure 5.7. (note: $I_R$ is the dilatancy
indicator in degrees and $Q$ is the natural logarithm of the grain crushing strength expressed in kilopascals). The friction angle, $\phi'$, ranges from 37.3° to 35.1°, with the corresponding dilation angle, $\psi$, between 7.9° and 5.1°, with $q_{\text{peak}}$ increasing from 237.3 to 759.0 kPa. The tendency is consistent with gradual suppression of dilatancy under increasing confining stress. This analysis provides further evidence that the incorporation of Bolton’s (1986) stress-dilatancy relationships helps the failure stress dependent model to capture the response of the sand in a realistic manner.

### 5.4.2 Failure mechanism at peak resistance

Figure 5.8 shows the normalised incremental deviatoric shear strain contours for spudcan S13 and the conical footings at peak resistance for tests with similar $H/D$. The incremental deviatoric strains are normalised by the maximum deviatoric shear strain to allow direct comparison of data from all tests in a clear manner. For each test a series of 10 images, covering a footing displacement of ~0.01D at peak resistance was analysed.

In all tests, a block of sand is pushed down to the underlying clay with inclination of the sand block at an angle similar to the back-calculated dilation angle given in Figure 5.7 (with the inclination angle of the failure plane – taken as equal to $\psi$ – superimposed on the PIV images). This further validates the assumption in the modified failure stress dependent model that the inclination angle of the failure mechanism should be taken as equal to the dilation angle of the sand at peak resistance, allowing the shape of the failure mechanism to vary dependent on the sand properties. In contrast, current industry guideline (ISO 2012) recommends the load spread method to calculate $q_{\text{peak}}$, where the load is assumed to spread through the upper sand layer to an imaginary footing of increased size at the sand-clay interface with an arbitrary projected angle. The PIV analyses in Figure 5.8 provide physical evidence that a failure stress dependent
approach is a more realistic and rational approach for predicting peak resistance than load spread method with seemingly arbitrary fixed geometries.

![Figure 5.8. Normalised incremental deviatoric shear strain contours at $q_{\text{peak}}$](image)

### 5.5 RESPONSE IN CLAY LAYER

#### 5.5.1 Transition from punch-through to meta-stable penetration

Figure 5.9(a) presents the back-calculated normalised bearing capacity factor from test H5C0 alongside the prediction of Equation 5.7. Figure 5.9b, c and d demonstrate the corresponding normalised incremental deviatoric shear strain contours at penetration depths of $0D$, $0.5D$ and $1.0D$ beyond the sand-clay interface, equivalent to $d/D = 0.6$, 1.1 and 1.6 respectively. For each stage a series of 10 images, covering a footing
displacement of ~0.01\(D\) was analysed. Figure 5.9b–d were combined to investigate the impact of the transition of deformation mechanisms from punch-through to meta-stable penetration on the bearing capacity factor.

![Figure 5.9](image)

**Figure 5.9.** (a) back-calculated bearing capacity factor; and normalised incremental deviatoric shear strain distribution (%) for test H5C0 at depths of (b) 0.6\(D\), (c) 1.1\(D\) and (d) 1.6\(D\)

When the footing was penetrated to the original sand-clay interface (Figure 5.9b), a significant amount of sand beneath the footing is undergoing shearing, as the sand plug is not yet fully formed. For the soils in this study, the sand is shearing in a drained...
manner and is thus likely to be much stiffer and stronger than clay shearing in an undrained manner at a similar depth, due to the strength of sand being stress dependent in drained shear. Furthermore, a conical wedge of clay is trapped beneath the sand plug, enlarging the soil flow mechanism further. The combination of these factors causes the back-calculated bearing capacity factor to be greater than that estimated by Equation 5.7.

As the footing and sand plug penetrate to the depth of $d/D = 1.1$, the back-calculated bearing capacity factor and Equation 5.7 start to converge as less sand is now shearing at the periphery of the trapped sand plug (Figure 5.9c), though the trapped cone of clay beneath the sand plug is still evident. At a penetration depth of $d/D = 1.6$ (Figure 5.9d), the soil flow mechanism consists of clay flowing around a composite foundation consisting of the footing and an entrapped meta-stable sand plug. The back-calculated bearing capacity factor response and Equation 5.7 converge as the shape of this composite foundation becomes meta-stable. Similar mechanistic behaviour was seen in other footing models of differing geometry, giving confidence that the Equation 5.7 is generally applicable, so along as the depth when $q_{\text{peak}}$ is equal to $q_{\text{clay}}$ is greater than or equal to $1D$ below the sand-clay interface.

### 5.5.2 Trapped sand plug geometry

In Equation 5.6 the sand plug shape is idealised as a cylindrical plug. For footings with similar $H/D$ but varying conical angle (ranging from 0 to 21°), the heights of the plugs, $H_{\text{plug}}$, were measured to account for the buoyancy effect when estimating the penetration resistance in the clay layer. Sand plug heights were measured directly from the photographs recorded during penetration testing, with all measurements converted from pixels to model scale.
Figure 5.10. Normalised sand plug height at various penetration depth (note: previous experimental data is for 1D below interface)

Figure 5.10 presents the sand plug heights after 0D, 0.5D and 1.0D penetration into the clay layer ($d/D = 0.6, 1.1$ and $1.6$ respectively), which facilitates investigation into potential differences in evolution of sand plug height with penetration depth for different footing shapes. The measurements of sand plug height from the centrifuge tests are presented alongside the experimental results (after 1.0D penetration into the clay layer) for footing S13 on medium dense sand (Hu et al. 2014a) for comparison. Overall, 7 tests in this study exhibit stable sand plug height throughout the footing penetration and reasonably fit the best-fit relationship of $H_{plug} = 0.9H_s$ resulting from previous analyses (Hu et al. 2014b) on a spudcan with 13° underside. This suggests that the friction on the footing-sand interface underneath the footing is sufficient such that slippage cannot occur, irrespective of the inclination angle of the underside of the footing. Therefore, the sand plug height should be taken as a constant of $0.9H_s$ for
simplicity, since no trend is evident in Figure 5.10 with respect to $H_s/D$ or the conical angle of the underside of the footing.

5.5.3 Bearing capacity factor model

Centrifuge tests on punch-through on sand overlying clay were collated from the literature, yielding a dataset that incorporated very dense ($I_D = 92\text{~}99\%$) and medium dense ($I_D = 43\text{ and }74\%$) sand centrifuge tests. Figure 5.11 presents the values of $N_c$ that were back-calculated from the experimental penetration resistance profiles. These $N_c$ values are higher than the range of 9-14 typical for spudcans penetration in single clay layer with linearly increasing shear strength with depth (Houlsby and Martin 2003; Hossain and Randolph 2009).

![Figure 5.11. Bearing capacity factors from very dense and medium dense sand centrifuge tests](image)

Larger sand layer height results in greater sand plug height and larger vertical sand-clay shear planes in the clay, enlarging the soil flow mechanism and increasing bearing
capacity factor, as is reflected in the form of Equation 5.7. Equation 5.7 was mainly derived from numerical analyses on loose and medium dense sand and only a single footing geometry, S13, was studied. Though the linear relationship from Equation 5.7 tends to underestimate $N_c$ at smaller $H_s/D$ and overestimate $N_c$ at larger $H_s/D$ moderately, the equation adequately estimates the bearing capacity factor of footing shapes ranging from flat to conical based with a conical angle no larger than 21° reasonably well. For footing with similar shape and $H_s/D$, the deep bearing capacity factor is not affected by the relative density of sand as once the sand plug is meta-stable the sand is not shearing as illustrated in Figure 5.9(d).

5.6 CONCLUSIONS

A series of model half-spudcan penetration tests have been reported that were conducted in the drum centrifuge. The effect of footing shape on the penetration resistance and associated soil deformation mechanisms during punch-through on sand overlying clay have been investigated. Photographs of soil flow mechanisms were captured in-flight and analysed using the PIV technique. Interpretation of this data has led to the following conclusions:

1. Flat footings mobilise significantly lower peak resistance than footings with a conical angle (angle of the base conical section as shown in Figure 5.2) no less than 7°. There is no systematic difference in $q_{peak}$ for footings with conical angle in the range of 7° and 21°.

2. The back-calculated distribution factors, $D_f$, were similar for footings with conical angles between 7° and 21° for similar $H_s/D$ ratio. At $q_{peak}$, larger conical angles result in increased accumulated radial strain near the outer radius of the footing. This is likely to invoke higher stress on the failure surface in the sand.
layer and would be expected to cause an increase in the $D_F$. However, proportionally higher accumulated deviatoric shear strains with increasing conical angle were also observed, causing significant softening of the sand in the same region and counteracting the effect of increased radial stress on the failure surface. This explains why the values of $D_F$ for footings with conical angles in the range of 7-21° are located in a narrow range.

3. Based on the back-calculated friction and dilation angles from a modified failure stress dependent model, larger $q_{\text{peak}}$ leads to lower values of $\phi'$ and $\psi$. This is consistent with the concept of gradual suppression of dilatancy under higher confining stress conditions during drained shearing. The normalised incremental deviatoric shear strain contours at $q_{\text{peak}}$ validate the assumption in the failure stress dependent model that the inclination angle of the trapped sand wedge may be taken as equal to the dilation angle of the sand at peak resistance.

4. The evolution of the soil flow mechanisms after penetration of the composite foundation into the underlying clay layer were observed using PIV. The bearing capacity factor reduced gradually to a steady state at penetration depths greater than $0.5D$ from the sand-clay interface. A constant sand plug height of $0.9H_s$ was confirmed to be an adequate approximation.

5. A linear equation can be used to estimate the bearing capacity factor of both spudcan and conical footings with conical angle $\leq 21^\circ$ reasonably well. The expression for bearing capacity is in a simple form, facilitating quick and easy evaluation of the potential for punch-through failure.

The conclusions provide further evidence of the benefit of incorporating the properties of sand into the analytical model of Lee et al. (2013b) and Hu et al. (2014a), when
compared to the methods currently provided to engineers in the ISO site assessment of
jack-up unit guideline (2012).

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CHAPTER 6. ASSESSING THE PUNCH-THROUGH HAZARD OF A SPUDCAN ON SAND OVERLYING CLAY

ABSTRACT

Rapid penetrations of offshore jack-up platform footings (spudcans) on a strong layer overlying a weak layer are still the most common cause of failure of a jack-up rig. Historically, at sites with a sand overlying clay stratigraphy, the safe installation of the jack-up rig is still hindered by the lack of accurate and robust design approaches for predicting the bearing capacity in both the overlying sand layer and the underlying clay layer. This paper describes numerical studies carried out to investigate the bearing capacity of spudcan foundations on sand overlying clay that utilise the Coupled Eulerian-Lagrangian (CEL) approach to accommodate large deformations. Modified Mohr-Coulomb and Tresca models are used to describe the sand and clay behaviour, with modifications accounting for the effects of strain softening on the response of the soil. The CEL results are shown to match centrifuge tests well allowing the study to be extended parametrically to cover a range of layer geometries, sand relative densities and footing shapes encompassing most cases of practical interest. An analytical model for predicting peak resistance is validated for medium dense to very dense sands and conical footing of angle 0° to 21°. The performance of an existing model to predict the peak resistance in the sand layer is assessed and an expression for the bearing capacity factor when the footing penetrates into the underlying clay is proposed based on back-calculations of all experimental and numerical investigations performed. A complete method to describe the full load-penetration resistance profile of a spudcan penetrating sand over clay is thus proposed. Comparisons of the full penetration resistance profile prediction method with centrifuge tests and field data show that the method provides a
reasonable first estimate of the punch-through distance; a basic reflection of the severity of a potential punch-through failure.

6.1 INTRODUCTION

Jack-up platforms are self-elevating mobile rigs, commonly employed to perform offshore oil and gas drilling. The advantage of their mobility has led to their use in a wide range of geographic and geotechnical conditions. Upon arrival at a new site, the footings of the jack-up platforms, known as spudcans, are preloaded to a vertical load that exceeds the maximum in-service load by using the weight of the rig and seawater ballast tanks. This serves as a proof of competence for resisting potential storm loads during operation. However, in a seabed consisting of sand overlying clay, there is a potential for punch-through failure during the installation. This is characterised by a rapid spudcan penetration of several meters in a few seconds (McClelland et al. 1983; Brennan et al. 2006; Kostelnik et al. 2007; Dean 2010). Such a failure may lead to damage or loss of the jack-up rig.

With punch-through being recognised as one of the most frequent hazards during spudcan installation and operation (Osborne and Paisley 2002), there is a need to improve the design approach for soil conditions liable to punch-through and to predict the severity of such an event. The punch-through distance (from onset of punch-through failure until equilibrium is re-established) of a punch-through event can be used as a basic indicator of severity and it can be characterised by two key phases: (i) the magnitude and depth of peak resistance in the sand layer and (ii) the depth at which the resistance in the underlying clay layer becomes equal to the peak resistance.
Several conceptual models (Hanna and Meyerhof 1980; Okamura et al. 1998; Teh 2007; Lee et al. 2013b) have been developed to estimate the peak resistance, $q_{\text{peak}}$ (in kPa), that occurs in the sand layer. Based on several series of centrifuge tests with medium dense to very dense sand overlying clay the failure-stress-dependent model of Lee et al. (2013b) was extended and further validated by Hu et al. (2014a) to predict $q_{\text{peak}}$ more accurately than the original model, for both medium dense and very dense sands, by accounting for mobilisation depth:

$$
q_{\text{peak}} = \left( N_{c0} s_{\text{um}} + q_0 + 0.12 \gamma_s' H_s \right) \left( 1 + \frac{1.76 H_s}{D} \tan \psi \right)^E \left( 1 + \frac{\gamma_s' D}{2 \tan \psi (E' + 1)} \left[ 1 - \left( 1 - \frac{1.76 H_s}{D} E \tan \psi \right) \left( 1 + \frac{1.76 H_s}{D} \tan \psi \right)^E \right] \right)
$$

where $N_{c0}$ is the bearing capacity factor for clay at the base of a circular foundation, which is obtained using the lower bound solutions proposed by Houlsby and Martin (2003); $s_{\text{um}}$ is the undrained shear strength of clay at sand-clay interface; $q_0$ is the effective overburden pressure at the surface; $\gamma_s'$ is the effective unit weight of sand; $H_s$ is the initial sand layer thickness; $D$ is the spudcan diameter; $\psi$ is the dilation angle of sand; and $E'$ is a parameter to simplify the algebra:

$$
E' = 2 \left[ 1 + D_F \left( \frac{\tan \phi^*}{\tan \psi} - 1 \right) \right]
$$

where $D_F$ is a distribution factor that relates the local stress along the failure surface to the average vertical stress, or the ratio of the normal effective stress at the slip surface to the mean vertical effective stress, and $\phi^*$ is a reduced friction angle caused by non-associated flow that can be expressed as (Drescher and Detournay, 1993):

$$
\tan \phi^* = \frac{\sin \phi \cos \psi}{1 - \sin \phi \sin \psi}
$$

where $\phi'$ is the operative friction angle of sand. Confirming the observations of Teh et al.
(2010), the depth (corresponding to the maximum bearing area) at which the peak resistance is mobilised was found experimentally to be ~0.12\(H\_s\) on average.

Equation 6.1 is limited to values of \(q_{\text{peak}}\) lower than the bearing pressure of the spudcan foundation in the sand alone, with methods for calculating the bearing pressure of circular footings in a pure sand layer described in, among others, Brinch Hansen (1970), Cassidy and Houlsby (1999, 2002), Randolph et al. (2004) and Lee (2009). In the design of the experimental programmes of Lee et al. (2013a) and Hu et al. (2014a, b, c), this limited the ratio of sand height to spudcan diameter \((H\_s/D)\) to less than 1. For higher ratios spudcans would remain in the sand layer under pressures typically experienced during jack-up preload (see discussion in Lee et al. 2013a). Therefore, the model of this paper is limited to the testing program used in its calibration, with \(H\_s/D < 1\).

When the spudcan penetrates through the sand layer into the clay layer, a sand plug is trapped underneath the spudcan (see Figure 6.1), effectively enlarging the footing size and mobilising clay with higher undrained shear strength under the base of the sand plug. The bearing capacity (at the spudcan depth) in the underlying clay can be expressed as:

\[
q_c = N_c s_{\text{u0}} + H_{\text{plug}} \gamma_c'
\]

where \(N_c\) is the bearing capacity factor in the clay layer (expressed in terms of the widest cross-sectional area of the footing); \(s_{\text{u0}}\) is the soil strength at the lowest elevation of the spudcan widest cross-sectional area; \(H_{\text{plug}}\) is the sand plug height, approximated as \(0.9H\_s\) (Teh 2007; Hu et al. 2014c); and \(\gamma_c'\) is the effective unit weight of the clay. The first term is the bearing capacity for a weightless soil, while the second term accounts for buoyancy. Although the clay beneath and around the sand plug is partially disturbed, the intact undrained strength of the clay at the widest cross-section of footing is used to
estimate the resistance in the clay layer. The effect of soil disturbance in the clay layer is essentially considered when fitting the expression of $N_c$. However, the equations for $q_{clay}$ were derived for a particular spudcan shape. The effect of footing shape combined with the relative density of the sand and shear strength of the clay was not considered.

**Figure 6.1. Schematic of spudcan foundation penetration in sand overlying clay**

The $N_c$ depends on the evolution of the sand plug geometry and classical expressions of $N_c$ for a single clay layer (Houlsby and Martin, 2003; Hossain et al. 2006) cannot consider the sand plug effect when predicting the penetration resistance of a spudcan following punch-through on sand overlying clay. Current ISO (2012) guidelines referred the equation proposed by Hossain et al. (2006), which is based on clearly different mechanisms to those observed in clay underlying sand layer using Particle Image Velocimetry (PIV) analysis (Teh et al. 2008; Hu et al. 2014b). Through wished-in-place analyses using the small strain finite element (FE) method, Lee (2009) proposed two equations to estimate bearing capacity factor corresponding to shallow and deep failure mechanisms (see Table 6.1). The sand plug height is a key input for
these equations since the resistance in the clay is taken as a function of the clay strength at the base of the sand plug. However, there was insufficient experimental or numerical evidence to confirm an appropriate height for the sand plug. As a result, a range of 0.6–0.9\( H_s \), was used by Lee (2009) to give lower and upper bound estimates on the penetration resistance. This rather wide range of sand plug height limits the accuracy of the full penetration resistance profiles predicted and the estimations of the potential punch-through distance. Lee et al. (2013a) suggested a linearly increasing \( N_c \) with \( H_s/D \), rather than sand plug height, from a series of very dense sand centrifuge tests on flat and spudcan footings. This relationship was improved by numerical and experimental studies incorporating both loose and medium dense sands overlying clay of varying strength in Hu et al. (2014c).

This paper reports the results of a large deformation FE study into the continuous penetrations of footings with different shapes (elevation view) on sand overlying clay conducted using the Coupled Eulerian-Lagrangian (CEL) approach available in the commercial package Abaqus/Explicit. The effect of footing shape with conical underside angle ranging from 0 to 21° has been investigated, which covers the range commonly used on jack-up rigs. This large deformation analysis method has the potential to provide a complete penetration resistance profile and to reveal the evolving soil flow patterns. A modified stress-level dependent Mohr-Coulomb model and a Tresca model considering strain softening were incorporated into the CEL approach via user subroutines for the sand and clay layers respectively. The CEL approach is first validated against centrifuge experimental data for different footing geometries, sand thicknesses and relative densities. Following this a comprehensive parametric analysis is developed for a realistic range of the geometrical and material parameters. The
performance of the simplified analytical approaches proposed in the literature for estimating the peak resistance in the sand layer and penetration resistance in the underlying clay layer are then appraised by comparison to the CEL simulations and existing testing data. The expression for the bearing capacity factor when the jack-up footing and trapped sand plug penetrate into the clay layer is updated based on all of the back-calculated experimental and numerical data. Finally the performance of the simplified analytical method is assessed via comparison to existing centrifuge and field test data.

6.2 NUMERICAL METHODOLOGY

6.2.1 Coupled Eulerian-Lagrangian approach

Previous applications of the CEL approach for modelling continuous spudcan penetration can be found in Tho et al. (2012, 2013), Qiu and Henke (2011), Qiu and Grabe (2012) and Hu et al. (2014c). In the CEL approach, the spudcan and soil are discretised using Lagrangian and Eulerian mesh. The Lagrangian mesh is composed of 4-node linear tetrahedron elements while the Eulerian mesh is composed of 8-node linear hexahedron elements with reduced integration and hourglass control. The soil mesh remains unchanged throughout the analysis and the soil materials are allowed to flow in or out of each Eulerian element whose nodes have fixed locations. Consequently, mesh distortion (and thus computational non-convergence) is avoided even when the soil is subjected to large deformations. The surfaces or interfaces of the soil materials are tracked by computing the volume fraction of each material within each element. An Eulerian element can be occupied by more than one material simultaneously, while only one material is contained in a Lagrangian element. The footing is discretised with Lagrangian elements which move through the Eulerian mesh
and the penetration resistance is obtained through contact between the footing and soil materials. The frictional footing-sand contact was described by the Coulomb friction law, with the coefficient of friction equal to $\alpha \tan \phi_v$, where $\phi_v$ is the constant volume friction angle of the sand and $\alpha$ is the footing roughness factor. Following the SNAME (2008) guideline, a roughness factor of $\alpha = 0.5$ was adopted, which was also the value used by Qiu and Henke (2011) and Qiu and Grabe (2012) in similar analyses. Additionally, it was necessary to define a layer of initially void elements above the mudline to accommodate the soil heave formed by footing penetration.

![Finite element mesh used in a typical CEL analysis (front view and note: $H_{\text{void}}$ and $H_{\text{clay}}$ vary with footing diameter and penetration depth)](image)

The penetration of the footing into the soil was simulated in displacement-controlled mode. A constant penetration velocity was specified as 0.2 m/s. After trialling various
element sizes, the element size around the footing was selected as $0.025D$ to achieve adequate numerical accuracy and efficiency. The dependency of the solution on the mesh density and penetration velocity has been discussed previously in Hu et al. (2014c). With the aim of minimising the computational cost and by taking advantage of the axial symmetry of the geometry of the problem, only a quarter of the domain was modelled as depicted in Figure 6.2. A fine mesh zone with horizontal extension of $0.75D$ was chosen to cover significant soil movements induced by footing penetration. The coarse mesh zone with horizontal extension of $2.5D$ was found to be sufficiently large to minimise the boundary effects. The depth of penetration, $d$, is defined as zero when the lowest elevation of the widest cross-sectional area of the conical footing or spudcan reaches the original soil surface.

6.2.2 Sand model

Considering the spudcan penetration rate in the experimental investigations (Hu et al. 2014a, b), it is reasonable to assume that the sand was under drained conditions and the clay was under undrained conditions. Tho et al. (2012) and Hu et al. (2014c) used the Mohr-Coulomb (MC) model to represent the mechanical response of sand to spudcan penetration. In its most basic form the MC model does not consider the effect of hardening-softening shear strength or variation of the dilatancy of sand, which are particularly critical when modelling the response of dense sand. The behaviour of dense sand was modelled more realistically by Qiu and Henke (2011) and Qiu and Grabe (2012) who used a hypoplastic model. However, this model needs 16 material properties and 13 state variables and, as a result, it is arduous to calibrate the required parameters and in the authors’ experience is difficult to maintain computational stability during long-distance penetration.
Table 6.1 Expressions for bearing capacity factors found in literature for spudcan foundations penetrating the underlying clay (after penetrating the top sand layer)

<table>
<thead>
<tr>
<th>Reference</th>
<th>Equation(s)</th>
<th>Parameter</th>
<th>Remark</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lee (2009)</td>
<td>$N_c$ for shallow failure mechanism: $N_c = 4 \frac{d_{base}}{D} + 9$; $\frac{d_{base}}{D} \geq \frac{H_{sh}}{D}$</td>
<td>$d_{base}$: the depth of the composite foundation from the sand-clay interface; $D$: diameter of the foundation; $k$: $\rho D s_{sm}$; $\rho$: strength gradient of clay; $s_{sm}$: clay shear strength at sand-clay interface; $H_{sh}$: the height of the composite foundation</td>
<td>Based on wished-in-place small strain analysis in which the soil deformation during continuous spudcan penetration was not considered. The sand plug height is critical in the equation since it correlates the clay strength at the bottom of the composite foundation to the resistance in clay. However, there were no sufficient experimental and numerical evidences showing the magnitudes of sand plug height. The values of $0.6H_s - 0.9H_s$ were assumed in the calculation.</td>
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<tr>
<td></td>
<td>$N_c$ for deep failure mechanism: $N_c = \left[ 1 - \frac{1}{2} k \frac{H_{sh}}{D} \right] \left( 18.2 \sqrt{\frac{H_{sh}}{D}} - 2 \right)$; $\frac{H_{sh}}{D} \leq 1.2$ and $\frac{d_{base}}{D} &gt; \frac{H_{sh}}{D} + 0.5$</td>
<td>$d_{base}$: the depth of the composite foundation from the sand-clay interface; $D$: diameter of the foundation; $k$: $\rho D s_{sm}$; $\rho$: strength gradient of clay; $s_{sm}$: clay shear strength at sand-clay interface; $H_{sh}$: the height of the composite foundation</td>
<td></td>
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<tr>
<td>Teh (2007)*</td>
<td>$N_c = 10(1 + 0.075) D \frac{h_{plug}}{D}$ $N_c \leq 11.5$</td>
<td>$d$: the spudcan penetration depth; $h_{plug}$: height of the trapped sand plug; $D$: diameter of the foundation</td>
<td>Derived for the case of spudcan penetration in single layer clay when soil back flow was initiated. The penetration mechanism with sand plug was not considered.</td>
</tr>
<tr>
<td>Lee et al. (2013b)</td>
<td>$N_c = 14 \frac{H_s}{D} + 9.5 \left( 0.21 \leq \frac{H_s}{D} \leq 1.12 \right)$</td>
<td>$H_s$: sand thickness; $D$: diameter of the foundation</td>
<td>Summarized from dense sand centrifuge test results on flat footings and spudcans. The effect of footing shape and relative density of sand were not analysed.</td>
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<tr>
<td>Hu et al. (2014c)</td>
<td>$N_c = 13 \frac{H_s}{D} + 9 \left( 0.16 \leq \frac{H_s}{D} \leq 1 \right)$</td>
<td>$H_s$: sand thickness; $D$: diameter of the foundation</td>
<td>Summarized from numerical analyses on loose and medium dense sand. Only one spudcan shape with 13° bottom shoulder angle was used in the simulation. The effect of footing shape and relative denser sand were not considered.</td>
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Note: * This formula was used in Teh (2007) and Teh’s method was referred in section A.9.3.2.6.4 and Clause E.3 of the ISO (2012) guideline.
### Table 6.2 Summary of centrifuge tests and numerical simulations

<table>
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<tr>
<th>Reference</th>
<th>#</th>
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<td>Hₐ (m)</td>
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## Assessing the Punch-through Hazard of a Spudcan on Sand Overlying Clay

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**Parametric Study**

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</table>

**Note:**
1. The bearing capacity factors corresponding to tests are from CEL.
2. * the value of $N_c$ (at 1D below the interface) is not available.
To overcome the above drawbacks and to describe the shearing behaviour of medium dense or very dense sands, the traditional MC model was modified by varying the internal friction angle, $\phi$, and dilation angle, $\psi$, with respect to the accumulated plastic shear strain, $\zeta$, as shown schematically in Figure 6.3. It was assumed that the friction angle increases linearly from an initial value, $\phi_{ini}$, to a peak value $\phi_p$ before reducing linearly to $\phi_{cv}$ when the critical state is approached. The threshold plastic shear strains corresponding to peak friction angle and critical state are denoted as $\zeta_p$ and $\zeta_{cv}$, respectively. The dilation angle remains zero when $\zeta \leq 1\%$ and then increases quickly to a peak value, $\psi_p$, at $\zeta = 1.2\%$. The dilation angle then remains at $\psi_p$ until $\zeta_p$, followed by linear reduction back to zero by $\zeta_{cv}$. The dilation angle is simplified as zero while $\zeta \leq 1\%$ since almost all sands are initially contractile before becoming dilatant, however, the MC model cannot converge with negative dilation angles. The incremental plastic shear strain during each incremental step was calculated as:

$$
\Delta \zeta = \sqrt{2\left[\left(\Delta \varepsilon_1 - \Delta \varepsilon_2\right)^2 + \left(\Delta \varepsilon_2 - \Delta \varepsilon_3\right)^2 + \left(\Delta \varepsilon_3 - \Delta \varepsilon_1\right)^2\right]/3}
$$

(6.5)

where $\Delta \varepsilon_1$, $\Delta \varepsilon_2$ and $\Delta \varepsilon_3$ are incremental principal plastic strains at the end of current step. This Modified Mohr-Coulomb (MMC) model is similar to that used by Potts et al. (1990; 1997), Dounias et al. (1996) and Troncone (2005). However, the implementation of this simple model was still within the framework of MC model: at the end of each incremental step the incremental plastic shear strain is calculated through Equation 6.5 and then the friction and dilation angles at each integration point are updated through the relationships in Figure 6.3 for the next step. The updated friction and dilation angles remain constant during the next step.
In the following analysis, the threshold shear strains were taken as $\varepsilon_p = 4\%$ and $\varepsilon_{cv} = 10\%$, as observed broadly in triaxial compression tests on super fine silica sand by Pucker et al. (2013). Similar super fine silica sands were used in centrifuge tests of Teh et al. (2008, 2010), Lee et al. (2013a) and Hu et al. (2014a). The initial friction angle $\phi_{ni}$ was taken as equal to $\phi_{cv}$. The peak friction angle $\phi_p$ is taken as a function of both relative density, $I_D$, and the mean effective stress applied on the soil, $p'$. The value of $p'$ in the penetrations of large-diameter footings is usually lower than 150 kPa (White et al. 2008). Bolton (1987) recommended the following equation to calculate dilatancy index, $I_R$, for $p' < 150$ kPa:

$$I_R = 5I_D - 1$$  \hspace{1cm} (6.6)

This equation together with another empirical formulation, Equation 6.7, proposed by Bolton (1986) are adopted for the calculation of $\phi_p$:

$$\phi_p = 3I_R + \phi_{cv}$$  \hspace{1cm} (6.7)
Chapter 6: Assessing the punch-through hazard of a spudcan on sand overlying clay

For the super fine silica sand used in the centrifuge tests by Lee et al. (2013a) and Hu et al. (2014a), $\phi_v = 31^\circ$. The peak dilation angle, $\psi_p$, is also estimated based on empirical relationship for axisymmetric problems (Bolton 1986):

$$\psi_p = \left(\phi_v - \phi_c\right) / 0.48$$  \hspace{1cm} (6.8)

The Young’s moduli of medium dense and dense sand were taken as 25 and 50 MPa, respectively. By incorporating this model in the simulation, the post-peak response of the sand is effectively captured in a computationally practical manner.

For the case of loose sand ($I_D \leq 35\%$), a hardening instead of softening behaviour is observed under drained conditions and the peak friction angle tends to be similar to or slightly higher than $\phi_v$. In this instance the modified model reverts to the traditional variant with the internal friction angle specified as the critical value and the corresponding dilation angle as zero.

### 6.2.3 Clay model

The clay layer under undrained conditions was modelled as an elastic-perfectly plastic material obeying the Tresca yield criterion incorporating strain softening effects. Similar to the implementation of the MMC model in the sand layer, the undrained shear strength at the integration points representing clay were updated at the end of each incremental step according to the accumulated absolute plastic shear strain (Einav and Randolph, 2005):

$$s_u = \left[\delta_{rem} + \left(1 - \delta_{rem}\right)e^{-3\zeta_{95}/s_t}\right]s_{ui}$$  \hspace{1cm} (6.9)

where $s_u$ is the remoulded undrained shear strength; $\delta_{rem}$ denotes the ratio of fully remoulded and intact shear strengths (i.e., the inverse of the sensitivity, $S_t$); $\zeta_{95}$ represents the value of $\zeta$ required for the soil to undergo 95% remoulding, estimated in the range of 10 to 50 for marine clays (Zhou and Randolph 2009); and $s_{ui}$ is the intact...
undrained shear strength of the clay, which was usually measured from penetrometer (e.g., T-bar or cone) tests. The undrained strength updated through Equation 6.9 was kept constant during the calculations of the next step time. A typical Young’s modulus of 500\(s_u\) was used and the Poisson’s ratio was 0.49 to approximate constant volume shearing under undrained conditions.

6.3 NUMERICAL ANALYSIS

6.3.1 Performance of the MMC model and Tresca model

The performance of the MMC and Tresca constitutive models is firstly validated against a test with very dense sand overlying clay from Lee et al. (2013a). The sand sample was a fine silica sand with \(I_D = 92\%\), and the friction and dilation angles (estimated via Equations 6.6-6.8) were \(\phi_p = 41.8^\circ\), \(\phi_v = 31^\circ\), and \(\psi_p = 22.5^\circ\) (see Table 6.2). The clay shear strength at the sand-clay interface, \(s_{um}\), was 17.7 kPa and shear strength increased linearly with a gradient \(k = 2.1\) kPa/m, measured using T-bar penetrometer tests. Sensitivity was measured as \(S_t = 3\) and an intermediate value of 25 was assigned to \(\zeta_{95}\).

The performance of the MMC and MC models are compared in Figure 6.4, highlighting the enhancement provided by the MMC model.

When the MC model with peak friction and dilation angles was employed, the penetration resistance continues to increase monotonically and the punch-through failure observed in the centrifuge test is not captured. The penetration resistance is overestimated significantly since the strength degradation of sand during the post-peak stage was not described. If the friction angle in the MC model was taken as the one at critical state and the corresponding dilation angle was zero, the peak resistance in the sand layer is significantly underestimated and the punch-through potential was not predicted well. In contrast, the MMC model results in a reasonable reproduction of the
test. A sharp increase in penetration resistance to the peak value is observed, which is followed by a significant reduction of resistance during subsequent penetration in the sand layer. With further penetration in the clay layer, the resistance is recovered gradually due to the undrained strength of clay increasing with depth. The difference in peak resistance predicted is within 10%, and the experimental and numerical resistance profiles in the clay layer agree well with each other.

In order to obtain more confidence in the application of MMC and modified Tresca models, three series of centrifuge tests with various soil properties and footing geometries were simulated. These consisted of 5 tests in very dense sands from Lee (2009), 9 tests in very dense or medium dense sands from Teh et al. (2010) and 11 tests
in medium dense sands in Hu et al. (2014b). The details of these tests and all the parameters are provided in Table 6.2.

![Graphs](image)

**Figure 6.5. Full penetration resistance profiles from centrifuge tests and numerical analyses**

Among a total number of 25 tests, two typical tests from each testing series were presented in Figure 6.5 for clarity. The robustness of the CEL approach incorporating the constitutive models developed is further verified through the reasonable agreement between the experimental and numerical penetration resistance profiles. The peak resistances predicted are usually moderately lower than those measured, however, the differences are less than 15% for the majority of the cases. One possible reason for the
underestimation (especially for very dense sand) is that in the CEL simulations, the mean stress along the sand failure surface is typically moderately lower than 150 kPa when \( q_{\text{peak}} \) is mobilised, thus the \( \phi_p \) and \( \psi_p \) corresponding to such lower stress level will be higher than those calculated using Bolton’s equation (White et al. 2008). More recently, Andersen and Schjetne (2013) reported practical values of \( \phi_p \) and \( \psi_p \) at different confining stress levels according to a database with more than 500 triaxial compression tests on 54 different onshore and offshore sands.

![Figure 6.6. Confining stress contours at \( q_{\text{peak}} \) for test UWA_F3](image)

According to the magnitude of confining stress observed in the CEL analysis for test UWA_F3 in Figure 6.6, 50–110 kPa, the value of \( \phi_p = 45^\circ \), \( \psi_p = 19^\circ \) and \( \phi_c = 37^\circ \) were determined from the fitting curves of Andersen and Schjetne (2013). Though the prediction of \( q_{\text{peak}} \) is improved as shown in Figure 6.5(b) for the above parameters, they were not used in the parametric simulations. Reasons for this choice were: (i) Bolton’s equations provide fast, automated and acceptable estimation of the input parameters for
the CEL analyses. The confining stress levels tend to increase during further penetration after peak resistance; and (ii) the testing data in Andersen and Schjetne (2013) were scattered in a relative large range.

Based on Lee et al. (2013b), Hu et al. (2014a) presented an analytical model to estimate the peak resistance. The model was calibrated through experimental data from medium dense to very dense sands and for a spudcan geometry with main conical angle of 13°, and has been further validated for various footing shapes with conical angle ranging from 0 to 21° (Hu et al. 2014b). The peak resistances predicted by the analytical model and CEL analyses compared to published experimental measurements are plotted in Figure 6.7. Compared with the predictions from the analytical model by Hu et al. (2014a), most numerical and almost all experimental data are located within bounds of ±15%.

![Figure 6.7. Comparison of peak resistances from analytical model of Hu et al. (2014a) and that from centrifuge tests and numerical analyses](image-url)
6.3.2 Evolution of friction angles and penetration resistance

The evolution of the friction angle for a typical test (H5S13 in Table 6.2, for spudcan with conical angle of 13°) is shown in Figure 6.8, in order to illustrate distributions of mobilised friction angles in the sand at different penetration depths. The experimental and numerical resistance profiles are provided in Figure 6.5(d). When the bottom shoulder of the spudcan was fully embedded in the sand (i.e., \( d = 0 \)), only the sand beneath the shoulder was sheared to the critical state. The friction angle of the sand in the outer region was then gradually mobilised towards the peak value with further penetration. The horizontal extension of the influence zone is as large as 0.7\( D \) from the centreline of the spudcan in Figure 6.8(a). When the depth of peak resistance, \( q_{\text{peak}} \), is reached, the influence zone has propagated wider and deeper, with more sand mobilised to a depth of 0.8\( D \) (Figure 6.8b). For subsequent penetration from the depth of peak resistance, the bearing capacity of clay was enhanced. More sand flows back to the top of the spudcan, acting as surcharge and contributing to the reduction in total penetration resistance.

At the depth of the original sand-clay interface, the majority of the sand around the advancing spudcan has reached the critical state and the minimum magnitude of penetration resistance was observed.

6.3.3 Evolution of soil and sand plug geometry

In Hu et al. (2014b), 11 half-footing penetration tests (against a perspex window) were conducted to investigate the footing shape effect and to visualise the soil flow mechanisms using the PIV technique. For penetration from occurrence of \( q_{\text{peak}} \) to 1\( D \) below the original sand-clay interface (for test H5C0 in Table 6.2, flat footing), the soil displacement vectors interpreted through the PIV analysis are displayed on the left side of Figure 6.9.
Figure 6.8. Friction angle contours at typical penetration depths for simulation of centrifuge test HSS13 ($\phi_p = 39.1^\circ$, $\phi_{cv} = 31^\circ$)
Figure 6.9. Evolution of soil and sand plug with footing penetration for test H5C0 (sand thickness 4.91 m, footing diameter 8 m) (LHS: Experimental PIV; RHS: CEL analysis)

The water-sand and sand-clay layer interfaces are indicated by two lines in the left hand side of Figure 6.9. The flow mechanism cannot be generated in a similar way in the CEL analysis since the velocities at the Eulerian nodes may not represent the physical/Lagrangian movements (Dassault Systèmes, 2011). Therefore, the deformed geometries of the sand and clay layers and the contours of clay shear strength are shown on the right side of Figure 6.9. In general, the sand plug shape and layer interface
positions measured using the PIV and the numerical soil deformations are consistent with each other.

Although \( q_{\text{peak}} \) occurs at a relatively shallow depth in the sand layer, obvious movements appear in clay beneath the sand-clay interface in Figure 6.9(a). This deformation might be idealised as a sand frustum beneath the footing moving downward and causing mobilisation of surrounding clay. Hence \( q_{\text{peak}} \) is the sum of the frictional resistance along the slip surface in the sand (see Figure 6.6) and the bearing capacity of the underlying clay minus the weight of the sand frustum, as reflected in the analytical model of Hu et al. (2014a). A sand wedge of truncated conical shape is pushed into the clay layer when the footing penetrates to the original sand-clay interface (Figure 6.9b).

The indentation of the sand wedge into the underlying clay squeezes clay outwards but the sand plug is not yet fully formed. Deeper soil with higher undrained strength is mobilised by the downward-moving sand wedge, resulting in increased penetration resistance. At 0.5\( D \) below the interface (Figure 6.9c), the sand plug is fully formed and its shape and height remained nearly unchanged as it is advanced further in the clay.

The penetration resistance is exerted by stronger clay at the sand plug base rather than at the spudcan bottom. The top layer sand continues to fall into the cavity formed above the footing. When the footing penetrates to 1\( D \) below the interface in Figure 6.9(d), the failure mechanism stabilises. For the underlying clay layer, the movement of the clay changes from vertical to radial and finally tails off to horizontal flow in the far field. Minor soil heave is found in the far-field of the soil surface, but the magnitude of soil upheaval measured in the present study is relatively small. The stable sand plug height underneath this flat footing is measured as 0.89\( H_s \).
The deformed soil layers corresponding to footings with conical angle of 14° and 21° (test H3C14 and H3C21), are shown in Figure 6.10. The stable sand plug height from the numerical simulations agree well with those measured in the centrifuge tests, with height of $0.96H_s$ for 14° angle and $0.98H_s$ for 21° angle. By comparing Figure 6.9d and Figure 6.10, the sand plug height increases slightly in proportion with the increase in footing conical angle but the effect is minimal. In all the PIV measurements and corresponding CEL analyses, the bottom of the sand plug has a relatively flat surface and its width is approximately equivalent to the diameter of the footing. Therefore, it is appropriate to assume that the sand plug is cylindrical in shape with diameter, $D$, and height, $H_{plug}$, when constructing a simplified method to calculate the bearing capacity in the clay layer.

![Figure 6.10. Sand plug heights for tests H3C14 and H3C21 (1D below sand-clay interface)](image-url)
6.4 BEARING CAPACITY FACTOR AND PUNCH-THROUGH DISTANCE

6.4.1 Bearing capacity factor in clay layer

In addition to the 25 centrifuge tests (see Table 6.2) replicated, a total number of 14 supplemental simulations were performed to back-calculate the bearing capacity factor, $N_c$, in the clay layer to further quantify possible combined influence of critical factors (e.g., footing shape, $H_s/D$, $I_D$, $s_{um}$ and $k$) within realistic bounds relevant to offshore practice. The sand properties were taken to be the same as the super fine silica sand used in the centrifuge tests in Lee et al. (2013a) and Hu et al. (2014a), with $I_D = 43\%$, $74\%$ and $85\%$; the sand thickness ratio $H_s/D$ is varied from 0.3 to 0.8; clay shear strength at the sand-clay interface $s_{um}$ ranges from 10 to 40 kPa; clay strength gradient $k$ is between 1.5 and 2.5 kPa/m; and the footing conical angle varies from $3^\circ$ to $21^\circ$. A typical value of 3 for soil sensitivity of kaolin clay is used, which is close to the measurements of Zhang et al. (2011) and Gan et al. (2012). The aim of selecting the above parameters is to cover more footing shapes and soil properties that have not appeared in the centrifuge tests performed. The details of the geometric and soil property parameters and the resulting $N_c$ are listed in Table 6.2. For the combined experimental and numerical dataset constituting all cases in Table 6.2 and the experimental and numerical data of Lee et al. (2013a) and Hu et al. (2014c), the variation of $N_c$ with the sand thickness ratio and normalised clay strength is shown in Figure 6.11. This combined dataset consists of 51 centrifuge tests and 60 numerical analyses cases. There is no obvious trend of $N_c$ depending on the normalised clay strength, $kD/s_{um}$. An equation that fits all these experimental and numerical data is:

$$N_c = 11 \frac{H_s}{D} + 10.5 \quad \left(0.16 \leq \frac{H_s}{D} \leq 1.12\right) \quad (6.10)$$
Figure 6.11. Bearing capacity factors from all the experimental and numerical analyses

Equation 4.8 is also plotted in Figure 6.11 for comparison, it can be seen that it fits the current trend but presents obvious discrepancy with the increase of $H_s/D$. The reason is that Equation 4.8 is solely based on numerical simulations and the majority of the cases have $H_s/D \leq 0.6$.

It is extremely difficult to establish the mechanical properties of the soils with complete certainty and the uncertainty originates from the natural variability of soils, measurement errors, imperfect interpretation models and insufficient data, etc (Lacasse et al. 2007). Statistical analysis is thus performed to quantify the uncertainties and scatter of the data in order to account for them in a rational and consistent manner that may be applied in design to give upper and lower bounds on the penetration resistance in the clay layer. As shown in Figure 6.11, the variability of $N_c$ is most easily expressed by assuming that it follows a normal distribution with a best fit represented by its mean.
(calculated from Equation 6.10) and bounds represented by its standard deviation. The corresponding coefficient of variation (COV) is 7.5% indicating low variation. This facilitates establishment of a range of punch-through distance predictions for spudcans on sand overlying clay, which will be discussed in the next section. The expression for bearing capacity factor is verified by comparison with those from all the centrifuge tests, plotting according to the relative density of sand and footing conical angle in Figure 6.12. As expected, though the data points are somewhat scatter around the best-fit line of Equation 6.10, neither the relative density of the sand or the footing conical angle has a significant or systematic effect on $N_c$. In the application of Equation 6.10, the standard deviation of the fitting equation can be added or subtracted from the best-fit line to yield bounds that cover the variability characterised by the scatter of the data points for the various footing geometries and soil conditions.

Figure 6.12. Comparison of the new expression for $N_c$ with values back-calculated from centrifuge tests

### 6.4.2 Prediction of full resistance profile and punch-through distance

Hu et al. (2014c) presented a simplified method to construct full resistance profile, in which the peak resistance is calculated by Equation 6.1 and the corresponding depth of
occurrence is $0.12H_s$. The bearing capacity factor in clay is updated currently to Equation 6.10, with the resistance in clay layer obtained by substituting Equation 6.10 into Equation 6.4. The punch-through distance is calculated as the distance from the depth of $q_{\text{peak}}$ to the depth where $q_{\text{clay}}$ equals to $q_{\text{peak}}$ in the underlying clay layer. The equations for calculating the penetration resistances in the full profile have been coded in a Microsoft Excel spreadsheet for quick application. The predictions based on the best fit equation, Equation 6.10, is appropriate for a basic evaluation. To the authors’ knowledge, almost all the sand overlying clay centrifuge tests reported in the literature in which punch-through failure were observed are listed in Table 6.2. The punch-through distance predicted through Equations 6.1, 6.4 and 6.10 are presented in Figure 6.13. The majority of the predictions of the punch-through distance are within $\pm20\%$ of the measured ones. The under-prediction of the punch-through distance may be partially due to the underestimation of $q_{\text{peak}}$ and partially due to the overestimation of the bearing capacity factor, which leads to larger penetration resistance in the underlying clay layer.

![Figure 6.13. Performance of the updated full profile prediction method](image)
While an accurate estimation of the actual penetration depth is required for jack-up installation, the use of statistical analysis on the bearing capacity factor, making reasonable allowance for scatter, is beneficial for desk-based risk assessment purposes. Calculating the mean together with ±1 standard deviation (σ) is often recommended within industry practise (Lacasse et al. 2007) (representing the 15.9 and 84.1 percentile of a normal distribution), though of course different confidence intervals could also be used (e.g. ±1.645σ representing the 5 and 95 percentile). Therefore, additional predictions on the punch-through distance based on the addition or subtraction of one standard deviation of $N_c$ provides an easy estimate of the range of uncertainty in the model prediction. For each spudcan penetration case, the calculation is repeated 3 times, with the $N_c$ adjusted according to the values specified by their statistical distribution.

Two typical centrifuge tests, H3C14 and D1SP40a in Table 6.2, are analysed to illustrate the potential of this simple statistical implementation, as demonstrated in Figure 6.14. The critical stages from the tests are captured from the predicted full profile and the simple prediction fits the testing curve reasonably well. The punch-through
distance measured in test D1SP40a is 11.83 m, compared with predicted value of 10.65 m through the mean value and ranged between 9.63 and 11.84 m through ±1σ. All the predictions of the punch-through distance based on this simple statistical treatment are summarised in Table 6.3.

Table 6.3. Predictions of punch-through distance based on best-fit expression and ±1 standard deviation for \( N_c \)

<table>
<thead>
<tr>
<th>Reference</th>
<th>#</th>
<th>Test name</th>
<th>( d_{\text{punch, test}} ) (m)</th>
<th>( \mu ) (m)</th>
<th>±1σ (m)</th>
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Note: All tests were conducted in the beam or drum centrifuge at the University of Western Australia (UWA).
6.4.3 Verification with field data

After comparison with a large number of centrifuge tests, the performance of the proposed method is further validated by back-analysing an in-field event reported by Baglioni et al. (1982). A three-legged jack-up mobile drill rig was installed in the North Sea. The spudcan footing had a diameter, $D$, of 10.67 m and a conical point extending approximately 6.10 m beneath the lowest elevation of the widest cross-sectional area of the spudcan. Though this is a very pointed spudcan and the conical angle is greater than the ones investigated (0~21°) previously, it should not have too much effect on the prediction of $q_{\text{peak}}$ as the $D_F$ values for various footing conical angles are prone to converge at high $H/D$. In-field investigation was performed by drilling one boring below the seafloor, the results of which are presented in the boring log shown in Figure 6.15a. This log shows that a fine to silty fine sand layer overlying clay, which caused the rig to punch-through when the leg loads approached the maximum preload condition of 60.31 MN. The punch-through distance was not reported.

Based on the spudcan dimensions and soil parameters in Figure 6.15a (the $I_D$ was estimated in between 35% and 85% to represent loose to medium dense sand and dense sand; $\phi_v$ was assumed as 31°), two full penetration resistance profiles are constructed in Figure 6.15b using the method developed here. These are shown together with the full profiles predicted by the method of Teh (2007), which is referred in the current industry guideline ISO (2012). ISO (2012) also suggested two additional methods to estimate the peak resistances only; the load spread method and punching shear method. The simplified analytical method developed incorporating Equation 6.10 predicts a maximum preload of 65.32 MN ($I_D = 85\%$) and minimum preload of 52.75 MN ($I_D = 35\%$), with corresponding punch-through distance of 11.66 and 8.76 m. These indicate a
severe punch-through failure is prone to occur. The peak resistance is underestimated significantly by the load spread and punching shear methods. Although the peak resistance is predicted reasonably by the method of Teh (2007), the potential for punch-through failure was not indicated.

![Figure 6.15. Prediction of full penetration resistance profile based on the parameters from field (h = horizontal; v = vertical)](image)

6.5 CONCLUSIONS

In this paper, the CEL approach was used to overcome the mesh distortion problem associated with numerical modelling of the penetration process of a spudcan into sand overlying clay. A Modified Mohr-Coulomb model and Tresca model were used to describe the behaviour of the sand and clay respectively. The numerical simulations show reasonable agreement with centrifuge tests on the very dense and medium dense
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sand, illustrating that the softening behaviour of the sand and clay can be properly simulated numerically. On the other hand, the traditional Mohr-Coulomb model for the sand does not well represent the punch-through features (especially for dense sand overlying clay cases) and should be used with caution.

The peak resistance from the experimental measurements and numerical simulations are within 15% of the predictions of the analytical model by Hu et al. (2014a), for loose to very dense sands and various footing shapes (conical angles of 0° to 21°). The evolution of the friction angle shows that the sand around the advancing spudcan reached the critical state during deep penetration. The soil movement was replicated through the numerical simulation and the sand plug heights fit the ones from the centrifuge tests very closely. The sand plug height is stable and nearly constant for penetration greater than 0.5D below the sand-clay interface. Footing conical angle does not have a significant effect on the sand plug height.

An expression for the bearing capacity factor is summarised based on a large experimental and numerical database incorporating various footing shapes and soil conditions. To account for the scatter in the data, a simple statistical method was proposed to provide mean, upper and lower bounds for the penetration resistance in the clay layer. The simplified prediction method of Hu et al. (2014c) is thus modified for estimating the bearing capacity in sand overlying clay, which enables a simplified bearing capacity–depth profile to be constructed. The majority of the predictions of the punch-through distance based on the above method fall within ±20% of those from comparable centrifuge tests. The comparison with field data highlights that the full profile prediction method has better performance over the methods referred in the current industry guidelines.
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6.6 REFERENCES


Chapter 6: Assessing the punch-through hazard of a spudcan on sand overlying clay


CHAPTER 7. A COMPARISON OF FULL PROFILE PREDICTION METHODS FOR A SPUDCAN PENETRATING SAND OVERLYING CLAY

ABSTRACT

Spudcans are the traditional footings used for offshore mobile jack-up rigs. Installation of spudcans in sand overlying clay can lead to punch-through failure, which can cause serious damage to the jack-up rig and endanger personnel. Accurate prediction of the profile of soil bearing capacity with depth is crucial to the offshore jack-up industry in evaluating the potential for and severity of punch-through failure. This note compares three new methods proposed in the literature and an interpretation of the existing International Standard Organisation (ISO) guideline for predicting the full penetration resistance profile. The penetration resistance profile for each of the methods is characterised by two key calculations: the peak resistance in the sand and the bearing capacity of the underlying clay. The punch-through distance – an indicator for the potential for and severity of punch-through failure – is estimated from these calculations. The methods are used to retrospectively simulate a database of 71 geotechnical centrifuge experiments of spudcans and conical footings penetrating sand-over-clay. The ISO guideline provides poor predictions, consistently underestimating the peak resistance in the sand and the underlying bearing capacity in the clay. Although all three new methods provide a superior response, by assessing the accuracy, scatter, and geometric skew of the predictions, the Teh method and Lee et al. method are shown to be biased with the normalised sand layer thickness $H/D$ in at least one of the key calculations used to define the penetration resistance profile, thus producing bias in the prediction of the punch-through distance. In contrast, the Hu et al. method yields largely
unbiased predictions and thus provides better predictions of the punch-through distance. Adoption of this method in the industry guidelines for site-specific assessment of jack-up operations could enable operators to better predict the occurrence of punch-through events.

7.1 INTRODUCTION

Offshore jack-ups typically consist of a buoyant triangular hull supported by three independent legs that are fitted with spudcan foundations (Figure 7.1). Punch-through events can occur during installation in sand overlying clay, when the sand layer yields causing the spudcan to plunge into the underlying weaker clay. In extreme cases, the rapid leg settlement causes structural damage to the legs or even toppling of the jack-up.

Figure 7.1. Problem definition and notation
Chapter 7: A comparison of full profile prediction methods for a spudcan penetrating sand overlying clay

A large number of centrifuge model tests investigating punch-through in sand overlying clay have been reported (Lee 2009; Teh et al. 2010; Lee et al. 2013a; Hu et al. 2014a, b). Retrospective simulation of this database is used to assess and compare punch-through predictions from three new methods, that of Teh (Teh 2007), Lee et al. (Lee 2009, Lee et al. 2013a, b) and Hu et al. (Hu et al. 2014a ~ d), as well as the latest industry guidelines for the site-assessment of jack-ups (ISO 2012).

7.2 SIMPLIFIED FULL PROFILE PREDICTION

All four methods simplify the spudcan penetration profile (Figure 7.2) as a combination of the: (a) spigot embedment resistance, (b) peak resistance in the sand layer $q_{\text{peak}}$, (c) resistance at the sand-clay interface, and (d) resistance in the clay layer $q_{\text{clay}}$. The punch-through distance is assessed by calculating the depth over which $q_{\text{peak}}$ is greater than $q_{\text{clay}}$.

![Figure 7.2. Simplified spudcan penetration resistance profile prediction method](image)

$q$ (kPa)

$d$ (m)
d_{\text{peak}}$A
B
C
D

A. spigot embedment B. $q_{\text{peak}}$ C. sand-clay interface D. $q_{\text{clay}} = q_{\text{peak}}$
Table 7.1 provides all of the design equations used to generate the predictions, with Figures 7.3 and 7.4 showing the basis of the calculations (for full descriptions see Teh 2007, Lee 2009, Lee et al. 2013b, Hu et al. 2014a ~ d and ISO 2012).

Table 7.1. Full profile design equations for the Teh method, Lee et al. method, ISO methods and Hu et al. method

<table>
<thead>
<tr>
<th>Method</th>
<th>Equations</th>
<th>Notation</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Teh (2007)</strong></td>
<td>$q_{pm} = \frac{n_{D} \cos \phi}{\tan \phi} W_{pm}$</td>
<td>$A$ the widest cross-sectional area of the spudcan</td>
</tr>
<tr>
<td></td>
<td>$Q_{pm} = \frac{n_{D} \cos \phi}{\tan \phi} (\phi - \delta) \left[ \tan \delta + \frac{1}{2} \rho_{c} \sin \phi \tan \phi \right]^{2}$</td>
<td>$d_{pm}$ depth from the bottom of the sand plug to the sand-face interface</td>
</tr>
<tr>
<td></td>
<td>$q_{pm} = \frac{n_{D} \cos \phi}{\tan \phi} \left( \frac{D_{p}}{2} \right)^{2}$</td>
<td>$d_{pm}$ depth factor</td>
</tr>
<tr>
<td></td>
<td>$D_{pm}$ spudcan depth at peak penetration resistance</td>
<td>$D_{pm}$ diameter of the spudcan</td>
</tr>
<tr>
<td></td>
<td>$D_{pm}$ distribution factor</td>
<td>$D_{pm}$ distribution factor</td>
</tr>
<tr>
<td></td>
<td>$E$ parameter to simplify the algebra</td>
<td>$E$ parameter to simplify the algebra</td>
</tr>
<tr>
<td></td>
<td>$h_{pm}$ distance between the depth of peak resistance and the sand-clay interface</td>
<td>$h_{pm}$ distance between the depth of peak resistance and the sand-clay interface</td>
</tr>
<tr>
<td></td>
<td>$H_{pm}$ height of the composite foundation of spudcan and sand plug</td>
<td>$H_{pm}$ height of the composite foundation of spudcan and sand plug</td>
</tr>
<tr>
<td></td>
<td>$H_{pm}$ sand plug height</td>
<td>$H_{pm}$ sand plug height</td>
</tr>
<tr>
<td></td>
<td>$k_{pm}$ sand thickness</td>
<td>$k_{pm}$ sand thickness</td>
</tr>
<tr>
<td></td>
<td>$\phi'$ strength gradient of clay</td>
<td>$\phi'$ strength gradient of clay</td>
</tr>
<tr>
<td></td>
<td>$K_{p}$ coefficient of passive earth pressure</td>
<td>$K_{p}$ coefficient of passive earth pressure</td>
</tr>
<tr>
<td></td>
<td>$K_{c}$ punching-shear coefficient</td>
<td>$K_{c}$ punching-shear coefficient</td>
</tr>
<tr>
<td></td>
<td>$n_{s}$ load-spread factor</td>
<td>$n_{s}$ load-spread factor</td>
</tr>
<tr>
<td></td>
<td>$N_{c}$ bearing capacity factor</td>
<td>$N_{c}$ bearing capacity factor</td>
</tr>
<tr>
<td></td>
<td>$N_{cr}$ bearing capacity factor of clay at base level of a circular foundation</td>
<td>$N_{cr}$ bearing capacity factor of clay at base level of a circular foundation</td>
</tr>
<tr>
<td></td>
<td>$q_{pm}$ effective overburden pressure</td>
<td>$q_{pm}$ effective overburden pressure</td>
</tr>
<tr>
<td></td>
<td>$q_{pp}$ peak penetration resistance</td>
<td>$q_{pp}$ peak penetration resistance</td>
</tr>
<tr>
<td></td>
<td>$E_{pm}$ the clay vertical bearing capacity subjected to</td>
<td>$E_{pm}$ the clay vertical bearing capacity subjected to</td>
</tr>
<tr>
<td></td>
<td>$q_{pm}$ vertical and inclined loadings within an area of radius $R$</td>
<td>$q_{pm}$ vertical and inclined loadings within an area of radius $R$</td>
</tr>
<tr>
<td><strong>Lee (2009) and Lee et al. (2013a, b)</strong></td>
<td>$q_{pm} = \frac{N_{c} + H_{cr}}{\tan \phi}$</td>
<td>$q_{pm} = \frac{N_{c} + H_{cr}}{\tan \phi}$</td>
</tr>
<tr>
<td></td>
<td>$q_{pm} = \frac{N_{c} + H_{cr}}{\tan \phi}$</td>
<td>$q_{pm} = \frac{N_{c} + H_{cr}}{\tan \phi}$</td>
</tr>
<tr>
<td></td>
<td>$N_{c} = \frac{H_{cr}}{\tan \phi}$</td>
<td>$N_{c} = \frac{H_{cr}}{\tan \phi}$</td>
</tr>
<tr>
<td></td>
<td>$H_{cr}$ complementary vertical bearing capacity at the spudcan level</td>
<td>$H_{cr}$ complementary vertical bearing capacity at the spudcan level</td>
</tr>
<tr>
<td></td>
<td>$\phi'$, $\phi''$, $\phi'''$</td>
<td>$\phi'$, $\phi''$, $\phi'''$</td>
</tr>
<tr>
<td></td>
<td>$\tau_{n}$ unsaturated shear strength of clay</td>
<td>$\tau_{n}$ unsaturated shear strength of clay</td>
</tr>
<tr>
<td></td>
<td>$\tau_{n}$ clay shear strength at lowest level of the spudcan</td>
<td>$\tau_{n}$ clay shear strength at lowest level of the spudcan</td>
</tr>
<tr>
<td></td>
<td>$H_{cr}$ average value of the shear strength from $d_{pm}$ to $H_{cr}$</td>
<td>$H_{cr}$ average value of the shear strength from $d_{pm}$ to $H_{cr}$</td>
</tr>
<tr>
<td></td>
<td>$\phi''$, $\phi'''$</td>
<td>$\phi''$, $\phi'''$</td>
</tr>
<tr>
<td></td>
<td>$\phi''$, $\phi'''$</td>
<td>$\phi''$, $\phi'''$</td>
</tr>
<tr>
<td><strong>ISO (2012)</strong></td>
<td>$q_{pm} = \frac{N_{c} + H_{cr}}{\tan \phi}$</td>
<td>$q_{pm} = \frac{N_{c} + H_{cr}}{\tan \phi}$</td>
</tr>
<tr>
<td></td>
<td>$q_{pm} = \frac{N_{c} + H_{cr}}{\tan \phi}$</td>
<td>$q_{pm} = \frac{N_{c} + H_{cr}}{\tan \phi}$</td>
</tr>
<tr>
<td></td>
<td>$N_{c} = \frac{H_{cr}}{\tan \phi}$</td>
<td>$N_{c} = \frac{H_{cr}}{\tan \phi}$</td>
</tr>
<tr>
<td></td>
<td>$H_{cr}$ complementary vertical bearing capacity at the spudcan level</td>
<td>$H_{cr}$ complementary vertical bearing capacity at the spudcan level</td>
</tr>
<tr>
<td></td>
<td>$\phi''$, $\phi'''$</td>
<td>$\phi''$, $\phi'''$</td>
</tr>
<tr>
<td></td>
<td>$\tau_{n}$ unsaturated shear strength of clay</td>
<td>$\tau_{n}$ unsaturated shear strength of clay</td>
</tr>
<tr>
<td></td>
<td>$\tau_{n}$ clay shear strength at lowest level of the spudcan</td>
<td>$\tau_{n}$ clay shear strength at lowest level of the spudcan</td>
</tr>
<tr>
<td></td>
<td>$H_{cr}$ average value of the shear strength from $d_{pm}$ to $H_{cr}$</td>
<td>$H_{cr}$ average value of the shear strength from $d_{pm}$ to $H_{cr}$</td>
</tr>
<tr>
<td></td>
<td>$\phi''$, $\phi'''$</td>
<td>$\phi''$, $\phi'''$</td>
</tr>
</tbody>
</table>

A comparison of full profile prediction methods for a spudcan penetrating sand overlying clay
7.2.1 Peak resistance

In the method of Teh, $q_{\text{peak}}$ is comprised of the vertical component of the shearing resistance in the mobilised sand frustum, the bearing capacity of the underlying clay, and the self-weight of the sand. However, the full bearing capacity of the underlying clay is assumed to act on a limited region from the centre line, beyond which the bearing capacity is assumed to reduce linearly to a minimum value of $0.5q_{\text{clay}}$ (Figure 7.3a). The reduction is due to the presence of shear stresses at the sand-clay interface incurred by horizontal outward movement of sand during failure (following Love et al. 1987).

The Lee et al. method assumes that $q_{\text{peak}}$ occurs when a sand frustum with a dispersion angle equal to the mobilised dilation angle of the sand is pushed into the underlying clay. The operational friction angle is determined from a modified form of the Bolton (1986) correlations. $q_{\text{peak}}$ is the sum of the frictional resistance in the sand, the bearing capacity of the underlying clay, and the weight of the sand frustum (Figure 7.3b). An empirical distribution factor $D_F$ relates the normal stress on the slip surface to the average vertical stress in the sand frustum. Expressions were back calibrated from 30 centrifuge tests on very dense silica sand.

The Hu et al. method modifies Lee et al. to account for the embedment depth attained during the mobilisation of $q_{\text{peak}}$ (Figure 7.3c) and extends it to various spudcan geometries and sand densities. Using an additional 15 centrifuge tests, the relationship for the distribution factor $D_F$ was optimised for the new mechanism, resulting in power relationships calibrated for both very dense and medium-dense sand and for footing conical angles from $0^\circ$ to $21^\circ$ (Hu et al. 2014a, b).

In all three methods the depth of the peak resistance $d_{\text{peak}}$ is taken as $0.12H_s$ (based on experimental observations of Teh et al. 2008, 2010; Hu et al. 2014a).
Chapter 7: A comparison of full profile prediction methods for a spudcan penetrating sand overlying clay

Shearing along logarithmic spiral failure surface

Sand
Clay
0.12 \( H_s \)
0.88 \( H_s \)
\( z \)
\( r \)
\( R \)
\( \psi \)
\( H_s \)
\( q_{\text{clay}} \)
Linearly reduced from \( q_{\text{clay}} \) to 0.5\( q_{\text{clay}} \)

(a) Teh method

(b) Lee method

(c) Hu et al. method

(d) Load spread method

(e) Punching shear method

Figure 7.3. Conceptual model for peak resistance from methods of (a) Teh; (b) Lee et al.; (c) Hu et al.; (d) Load spread method of ISO; and (e) Punching shear method of ISO

ISO (2012) recommends both load spread and punching shear methods. For the load spread method (Figure 7.3d), \( q_{\text{peak}} \) is equated to the capacity of a fictitious footing of increased area at the interface between the sand and clay layers (with the recommended load spread ratio of 3 used in this note as it has been verified in Lee et al. (2013b) and Figure 3.11 that a ratio of 3 always performs better than a ratio of 5 in predicting \( q_{\text{peak}} \)). However, there is ambiguity in ISO regarding the position of the surcharge during the
calculation of the baring capacity of the larger fictitious footing, as shown in Figure 7.3d. The ISO guideline (Figure A9.3-11 of ISO 19905.5) indicates the effective overpressure at the footing level should be used in conjunction with the bearing capacity equation A9.3-7. In this case the pressure should remain as just $q_0$ when calculating the bearing capacity of the fictitious footing on top of the clay layer. However, the authors interpret that the actual surcharge on the clay layer should be assumed. That is the surcharge at the spudcan level ($q_0$) should have the additional surcharge due to the sand layer between it and the fictitious added to it (i.e., $q_0 + \gamma_s H_s$). To provide a comparison both assumptions are used in this note to retrospectively predicting the experimental database. The two forms of the design equations are Equation 7.17a and 7.17b.

The punching shear method (Figure 7.3e) of the ISO guidelines assumes that a cylindrical frustrum of sand is pushed into the underlying clay, mobilising frictional resistance on the vertical surface of the frustrum and clay bearing capacity at the base. The frictional resistance is controlled by a punching shear coefficient, $K_s$, which can be derived from the design chart provided in ISO. The punching shear method has the same ambiguity as the load spread method regarding the position of the surcharge and therefore both assumptions are calculated (Equation 7.18a and 7.18b in Table 7.1).

7.2.2 Clay resistance

All four methods use a form of the bearing capacity equation to calculate spudcan resistance in the underlying clay layer, though with differing interpretations of $N_c$ and the influence of the sand plug that has been shown to become trapped beneath the spudcan (Craig and Chua 1990; Teh et al. 2008; Lee et al. 2013a).

All three new methods assume the entrapped sand plug is cylindrical in shape (Figure 7.4a ~ c), though a range of sand plug height from 0.6 to $0.9 H_s$ were measured. To fairly
compare the three methods, a value of $0.9H_s$ is used in the retrospective predictions. ISO (2012) does not mention a sand plug and therefore none is assumed.

![Diagram of soil bearing capacity](image)

**Figure 7.4. Nomenclature of soil bearing capacity in underlying clay for methods of: (a) Teh; (b) Lee et al.; (c) Hu et al.; (d) ISO**

Although acknowledging the increased footing size due to the sand plug, the Teh method still uses $N_c$ factor derived by Hossain et al. (2006) for just a spudcan penetrating into a single clay layer. The Lee et al. method uses new $N_c$ relations derived from small strain finite element (FE) analyses of buried cylinders (accounting for the composite spudcan and sand plug). Alternatively, the Hu et al. method simplifies $N_c$ by using an expression back-calculated from both centrifuge and large deformation FE analyses of the full penetration process. The ISO guideline provides some expressions for $N_c$ and the equations from Houlsby and Martin (2003) are used.
7.3 COMPARISON AND DISCUSSION

The 71 centrifuge tests contributing to the experimental database are detailed in Table 7.2. The footing conical angles range from 0° to 21°, the range of the normalised sand layer thickness is 0.16-1.12, and the relative density of sand \( \rho_D \) varies between 43% and 99%. The clay strength at sand-clay interface, \( s_{um} \), ranges from 7.22 kPa to 25.82 kPa, while the strength gradient \( k \) varies between 1.20 kPa/m and 2.13 kPa/m. These cover the range of parameters and geometries typically encountered in the field. Punch-through was observed in 62 of the 71 tests.

7.3.1 Full penetration-resistance profile predictions

The methods are evaluated using retrospective prediction of the centrifuge experiments. Four typical centrifuge tests, two very dense sand tests (\( \rho_D = 92\% \)) from Lee et al. (2013a) and two medium dense sand tests (\( \rho_D = 43\% \)) from Hu et al. (2014a), are highlighted here to evaluate the performance of each method. Predictions of the penetration resistance profile for the 66 tests (except 5 beam centrifuge test of Lee (2009), for which the penetration resistance profiles were not available) are provided as supplementary data.

As shown in Figure 7.5, both \( q_{peak} \) and \( q_{clay} \) predicted by the Teh method deviate from those measured, and no punch-through is predicted for some of the cases in which punch-through was observed in the experiments. In the cases shown in Figure 7.5, the Lee et al. method overestimates \( q_{clay} \), which causes poor prediction of the punch-through distance. Due to the adoption of load spread or punching shear method for the calculation of \( q_{peak} \) and the effect of the sand plug in the underlying clay layer being ignored, the ISO method underestimates the penetration resistances significantly. In contrast, the Hu et al. method displays better prediction of both \( q_{peak} \) and \( q_{clay} \), leading to
a better overall prediction of the penetration resistance profile and the punch-through distance.

![Graphs showing comparison of calculation methods](image)

**Figure 7.5.** Selected comparisons of experimentally measured punch-through profiles with all calculation methods (for test names refer to the supplementary data or Lee et al. 2013a and Hu et al. 2014a)
### Table 7.2. Summary of sand overlying clay centrifuge tests reported in the literature

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Centrifuge type</td>
<td>beam</td>
<td>beam</td>
<td>beam</td>
<td>drum</td>
<td>drum</td>
<td>drum</td>
<td>beam/drum</td>
</tr>
<tr>
<td>Number of tests</td>
<td>5</td>
<td>7</td>
<td>3</td>
<td>30</td>
<td>15</td>
<td>11</td>
<td>71</td>
</tr>
<tr>
<td>Geometry</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D (m)</td>
<td>8‒14</td>
<td>10</td>
<td>4‒8</td>
<td>6‒16</td>
<td>6‒20</td>
<td>8</td>
<td>4–20</td>
</tr>
<tr>
<td>H_s (m)</td>
<td>7</td>
<td>3‒10</td>
<td>3.5‒7.1</td>
<td>3.4‒6.7</td>
<td>3.2‒6</td>
<td>3.03‒7.25</td>
<td>3‒10</td>
</tr>
<tr>
<td>H_s/D</td>
<td>0.50‒0.88</td>
<td>0.3‒1.0</td>
<td>0.58‒0.89</td>
<td>0.21‒1.12</td>
<td>0.16‒1.00</td>
<td>0.38‒0.91</td>
<td>0.16‒1.12</td>
</tr>
<tr>
<td>Conical angle (°)</td>
<td>13</td>
<td>10</td>
<td>13</td>
<td>0–13</td>
<td>13</td>
<td>0–21</td>
<td>0–21</td>
</tr>
<tr>
<td>Sand</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>I_d (%)</td>
<td>99</td>
<td>58‒95</td>
<td>98‒99</td>
<td>92</td>
<td>43</td>
<td>74</td>
<td>43–99</td>
</tr>
<tr>
<td>$\gamma'_s$ (kN/m³)</td>
<td>11.15</td>
<td>9.15‒9.93</td>
<td>11.13‒11.15</td>
<td>10.99</td>
<td>9.96</td>
<td>10.61</td>
<td>9.15‒11.15</td>
</tr>
<tr>
<td>Clay</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>k (kPa/m)</td>
<td>1.85</td>
<td>1.56</td>
<td>1.20</td>
<td>2.10</td>
<td>1.54‒1.55</td>
<td>1.51‒2.13</td>
<td>1.2–2.13</td>
</tr>
<tr>
<td>$\gamma'_c$ (kN/m³)</td>
<td>N/A</td>
<td>6</td>
<td>6.5</td>
<td>7.5</td>
<td>7.11</td>
<td>7.21</td>
<td>6–7.5</td>
</tr>
<tr>
<td>Results</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$q_{\text{peak, test}}$ (kPa)</td>
<td>421.30–606.48</td>
<td>154.78‒699, 54</td>
<td>270–608</td>
<td>219–712</td>
<td>169.92‒382.95</td>
<td>237.28‒758.95</td>
<td>154.78‒758, 95</td>
</tr>
<tr>
<td>$d_{\text{punch, test}}$ (m)</td>
<td>N/A</td>
<td>3.93–7.30</td>
<td>9.16–10.33</td>
<td>0.20–12.20</td>
<td>5.22–8.10</td>
<td>4.75–13.94</td>
<td>0.20–13.94</td>
</tr>
</tbody>
</table>
Chapter 7: A comparison of full profile prediction methods for a spudcan penetrating sand overlying clay

\(H/D\) ratio and a linear regression line for each method is displayed to highlight the skew of the prediction. For the load spread and punching shear method of ISO, the interpretation with the additional surcharge (Equation 7.17b and 7.18b) was used in Figure 7.6 as it has been shown in Figure 7.5 that it performs better than its counterpart (Equation 7.17a and 7.18a) in the prediction of \(q_{\text{peak}}\). The ISO guidelines should be more clearly drafted to make this assumption less ambiguous. However, the comparisons of both interpretations can be found in Table 7.3, which summarises the performance of each method with statistical parameters.

Table 7.3. Model performance indicators for each of the prediction methods

<table>
<thead>
<tr>
<th>Method</th>
<th>Teh method</th>
<th>Lee method</th>
<th>Hu et al. method</th>
<th>ISO (2012)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(q_{\text{peak}, \text{calculated}}/q_{\text{peak, measured}})</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number of tests</td>
<td>71</td>
<td>71</td>
<td>71</td>
<td>71</td>
</tr>
<tr>
<td>Min.</td>
<td>0.57</td>
<td>0.77</td>
<td>0.83</td>
<td>0.07</td>
</tr>
<tr>
<td>Max.</td>
<td>1.42</td>
<td>1.28</td>
<td>1.39</td>
<td>0.53</td>
</tr>
<tr>
<td>Mean</td>
<td>0.86</td>
<td>0.98</td>
<td>1.01</td>
<td>0.29</td>
</tr>
<tr>
<td>(\sigma)</td>
<td>0.16</td>
<td>0.09</td>
<td>0.08</td>
<td>0.10</td>
</tr>
<tr>
<td>(\theta^* (\degree))</td>
<td>27.29</td>
<td>-4.01</td>
<td>1.47</td>
<td>-13.18</td>
</tr>
<tr>
<td>COV (%)</td>
<td>17.98</td>
<td>9.36</td>
<td>8.19</td>
<td>35.89</td>
</tr>
</tbody>
</table>

| \(N_{\text{c, calculated}}/N_{\text{c, measured}}\) |        |            |                  |            |
| Number of tests*        | 54       | 54         | 54               | 54         |
| Min.                    | 0.44      | 0.64       | 0.83             | 0.30       |
| Max.                    | 0.97      | 1.10       | 1.16             | 0.60       |
| Mean                    | 0.69      | 0.89       | 0.97             | 0.45       |
| \(\sigma\)              | 0.11      | 0.10       | 0.07             | 0.07       |
| \(\theta^* (\degree)\)  | -24.47    | -19.55     | 0.61             | -15.32     |
| COV (%)                 | 16.49     | 11.65      | 7.70             | 15.66      |

| \(d_{\text{punch, calculated}}/d_{\text{punch, measured}}\) |        |            |                  |            |
| Number of tests*        | 32       | 47         | 54               | 36         |
| Min.                    | 0.50      | 0.63       | 0.67             | 0.49       |
| Max.                    | 1.95      | 2.16       | 2.24             | 4.54       |
| Mean                    | 0.97      | 1.00       | 1.06             | 1.05       |
| \(\sigma\)              | 0.34      | 0.34       | 0.39             | 0.75       |
| \(\theta^* (\degree)\)  | 37.20     | -8.20      | 2.71             | -58.07     |
| COV (%)                 | 35.50     | 33.60      | 36.54            | 71.96      |

Note: The skew angle \(\theta^*\) is the arctangent of the gradient of the linear regression line (see Figure 7.6).

# Only 54 tests are available for \(N_{c}\) comparison (see supplementary Table 1).

* In total 62 tests have \(d_{\text{punch}}\) values (see supplementary Table 1). This row reflects the number of punch-through potential predicted from each method.
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As shown in Figure 7.6a, though the prediction of $q_{\text{peak}}$ is improved by the additional surcharge, both load spread method and punching shear method under-predict $q_{\text{peak}}$ significantly, with mean values of 0.58 and 0.59 respectively.

The load spread ratio $n_s$ (see Table 7.1) required to fit the experimental data has been back-calculated and is shown in Figure 7.7 to fit the range $0.91 < n_s < 2.07$, with a mean of 1.42. This is far smaller than the range between 3 and 5 recommended in the guideline. However, it is consistent with the actual spudcan penetration data in Baglioni et al. (1982), which also suggested smaller $n_s$ values.

The Teh method provides a conservative prediction of $q_{\text{peak}}$, with $q_{\text{peak, calculated}}/q_{\text{peak, measured}}$ generally less than unity. The predicted $q_{\text{peak}}$ values might be as low as 40% of the experimental measurements. This is the underlying cause of the inaccurate prediction of no punch-through in both Figure 7.5b and d and in 28 of the other tests presented in the supplementary data. A large variation of the predictions is indicated by a high coefficient of variation (COV) of 17.98%. The Lee et al. method yields a significantly improved prediction, with $q_{\text{peak, calculated}}/q_{\text{peak, measured}}$ between 0.77 and 1.28, a mean value of 0.98, and a much-reduced COV of 9.36%. However, both Figure 7.6a and Table 7.3 indicate that mild bias exists with $H_s/D$ with a skew angle of -4.01°. This is improved upon with the Hu et al. method providing reasonably good comparisons with $q_{\text{peak, calculated}}/q_{\text{peak, measured}}$ between 0.83 and 1.39, a mean value of 1.01, and the lowest scatter and skew of all the methods at 8.19% and 1.47° respectively.
Figure 7.6. Comparison of the performance of the calculation methods against centrifuge database: (a) $q_{\text{peak}}$; (b) bearing capacity factor $N_c$; and (c) punch-through distance
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Figure 7.6b shows that the adoption of $N_c$ values for spudcan penetration in single-layer clay causes significant under-prediction of $N_c$ in the Teh and ISO methods, worsening with increasing $H_s/D$. $N_c$ should increase with $H_s/D$ due to a larger entrapped sand plug and this bias is largely alleviated in the Lee et al. method because the composite footing and sand plug is accounted for. However, a certain amount of skew remains because the FE analyses, on which the $N_c$ factors are based, did not account for the drag-down of softer near-surface sediments or strain softening. The large deformation analyses performed by Hu et al. (2014c, d) accounted for both of these effects, resulting in better predictions of $N_c$ across the full range of $H_s/D$.

![Graph showing comparison of prediction methods](image)

**Figure 7.7.** Back-calculated $n_s$ for the load spread method of ISO (2012) to equal the experimentally measured $q_{peak}$ (note: the additional surcharge of Equation 7.17b is considered) (the legend references the experimental database)

Finally, the punch-through distance measured in the centrifuge tests is compared with those predicted. Although 62 tests were predicted to experience punch-through failure based on the ISO load spread method, this is considered a bit of a coincidence as
significant under-prediction of $q_{\text{peak}}$ and $N_c$ were observed. In addition, large scatter and skews are reflected with COV of 155.89% and skew angle of -79.83°. The situation was not improved for the ISO punching shear method, with a mean of 2.45 and corresponding COV of 146.95%. The Teh method could only capture punch-through for 32 tests out of the 62 tests. Significant scatter was indicated by a COV of 35.5%, while the minimum discrepancy of the punch-through distance was ~50%. The Lee et al. method performs reasonably well (correctly predicting punch-through in 47 cases) despite being based on a subset of the data used in the comparison, but certain cases remain that produce significant overestimations of the punch-through distance. Due to more accurate predictions of both $q_{\text{peak}}$ and $N_c$ (note: the Hu et al. method does not attempt to represent the penetration resistance between $q_{\text{peak}}$ and the sand-clay interface), the Hu et al. method predicts the majority of the punch-through cases (54 in total). The estimated punch-through distances are within ±20% for the majority of the cases, and the skew is the smallest of all the methods at 2.71°. It should be noted here that punch-through failure was coincidently predicted from the ISO methods due to both underestimation of $q_{\text{peak}}$ and $q_{\text{clay}}$, and a reasonable evaluation of each method should be made based on the combination of $q_{\text{peak}}$, $N_c$ and $d_{\text{punch}}$.

7.4 CONCLUSIONS

The performance of the existing ISO guideline and three new alternative methods for calculating the full resistance-penetration of a spudcan into sand overlying clay has been assessed. The ISO method is shown to provide worryingly inaccurate predictions for this 71 test database. This is because the methods include (i) inappropriate failure mechanisms for the peak resistance, and (ii) bearing capacity factors in the underlying clay that do not account for entrapped sand. Within the alternative methods, the Teh
method shows skew in the $q_{\text{peak}}$ and $N_c$ predictions with respect to $H_s/D$. In certain cases, this results in the method erroneously indicating no potential for punch-through failure. Although the Lee et al. method shows better performance overall, there is a certain amount of skew in the predictions with respect to $H_s/D$, and often, $q_{\text{peak}}$ is well predicted while $q_{\text{clay}}$ is poorly predicted or vice versa. The Hu et al. method shows the least skew with $H_s/D$, resulting in better predictions of $q_{\text{peak}}$, $N_c$ and thus $d_{\text{punch}}$ across the range of $H_s/D$ investigated (note: 49% of the dataset used in the comparison were used to derive the Hu et al. method). The constructed penetration-resistance profiles, which capture the distinctive aspects of a typical spudcan penetration resistance profile, are comparable with those obtained experimentally. Adoption of this method in the industry guidelines for jack-up operations could enable operators to better predict the occurrence of punch-through events. It should be noted that this method was summarised based on practical ranges of soil properties for offshore locations, where punch-through failure are potential risks. It may not be valid outside the bounds investigated here or for soils with significantly differing behaviours (e.g., highly sensitive clays or carbonate silts). Although the method has been tested on a comprehensive set of centrifuge tests, additional validation against field data would enhance confidence in its application.

7.5 REFERENCES


Chapter 7: A comparison of full profile prediction methods for a spudcan penetrating sand overlying clay


APPENDIX

Table 7a.1. Summary of footing penetration tests in the literature

<table>
<thead>
<tr>
<th>Investigation</th>
<th>Test Name</th>
<th>Geometry</th>
<th>Sand</th>
<th>Clay</th>
<th>Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lot (2009) UWA Tests</td>
<td>D1SP60a</td>
<td>13°</td>
<td>6.2</td>
<td>10</td>
<td>0.92</td>
</tr>
<tr>
<td>Teh et al. (2010) &amp; Teh (2007)</td>
<td>D1SP70a</td>
<td>13°</td>
<td>6.2</td>
<td>14</td>
<td>0.44</td>
</tr>
<tr>
<td></td>
<td>D1SP50a</td>
<td>13°</td>
<td>6.2</td>
<td>10</td>
<td>0.62</td>
</tr>
</tbody>
</table>

Note: *The full penetration resistance profiles are not available for this series of tests.** The penetration depth less than 1D below the sand-clay interface. ** Values are unavailable or not measured. * Test was reported in Teh et al. (2010), and load-penetration curve can be found in Teh (2007).
Figure 7a.1. Comparison of measured and predicted penetration resistance profile for (a) NUS_F1; (b) NUS_F2; (c) NUS_F3 and (d) NUS_F4
Figure 7a.2. Comparison of measured and predicted penetration resistance profile for (a) NUS_F5; (b) NUS_F8; (c) NUS_F9 and (d) UWA_F3
Figure 7a.3. Comparison of measured and predicted penetration resistance profile for (a) UWA_F4; (b) UWA_F10; (c) D1F30a and (d) D1F40a
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Figure 7a.4. Comparison of measured and predicted penetration resistance profile for (a) D1F50a; (b) D1F60a; (c) D1F70a and (d) D1F80a
Figure 7a.5. Comparison of measured and predicted penetration resistance profile for (a) D1F40b; (b) D1F50b; (c) D1F60b and (d) D2F30a.
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Figure 7a.6. Comparison of measured and predicted penetration resistance profile for (a) D2F40a; (b) D2F60a; (c) D2F80a and (d) D2F30b
Figure 7a.7. Comparison of measured and predicted penetration resistance profile for (a) D2F40b; (b) D2F60b; (c) D2F80b and (d) D2F30c
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Figure 7a.8. Comparison of measured and predicted penetration resistance profile for (a) D2F40c; (b) D2F60c; (c) D2F80c and (d) D2F30d
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Figure 7a.9. Comparison of measured and predicted penetration resistance profile for (a) D2F40d; (b) D2F60d; (c) D2F80d and (d) D1SP40a
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Figure 7a.10. Comparison of measured and predicted penetration resistance profile for (a) D1SP50a; (b) D1SP60a; (c) D1SP70a and (d) D1SP80a
Figure 7a.11. Comparison of measured and predicted penetration resistance profile for (a) L1SP1; (b) L1SP2; (c) L1SP3 and (d) L1SP4
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Figure 7.12. Comparison of measured and predicted penetration resistance profile for (a) L1SP5; (b) L2SP1; (c) L2SP2 and (d) L2SP3
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Figure 7a.13. Comparison of measured and predicted penetration resistance profile for (a) L2SP4; (b) L2SP5; (c) L3SP1 and (d) L3SP2
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Figure 7a.14. Comparison of measured and predicted penetration resistance profile for (a) L3SP3; (b) L3SP4; (c) L3SP5 and (d) H7C7
Figure 7a.15. Comparison of measured and predicted penetration resistance profile for (a) H7C14; (b) H7C21; (c) H5C0 and (d) H5C7
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Figure 7a.16. Comparison of measured and predicted penetration resistance profile for (a) H5S13; (b) H5C14; (c) H5C21 and (d) H3C7
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Figure 7a.17. Comparison of measured and predicted penetration resistance profile for (a) H3C14 and (b) H3C21
CHAPTER 8. CONCLUDING REMARKS

8.1 INTRODUCTION

This thesis has proposed a simplified full profile prediction method for assessing spudcan installation in sand overlying clay soils where punch-through failure is a potential hazard. Both physical and numerical modelling investigations were undertaken to amass a large test database covering many combinations of footing shapes, dimensions and soil properties. This database has then been used to test the analytical method developed to predict the penetration resistance. Particular attention was paid to the effect of the relative density of sand and its effect on dilation and shear strength, the shear strength of the underlying clay and the footing shapes on the peak resistance in the sand layer and the penetration resistance in the underlying clay layer.

This chapter concludes the main findings from this research, followed by recommendations for future work.

8.2 ORIGINAL CONTRIBUTIONS AND MAIN FINDINGS

8.2.1 Modified failure-stress-dependent model for the prediction of $q_{peak}$

A medium dense sand overlying clay sample was successfully formed in the drum centrifuge to investigate spudcan penetration behaviour, which has not previously been achieved at UWA and has rarely been reported in the literature. Chapter 3 presented a set of fifteen tests on three different sand layer heights with a range of footing sizes ($H/D$ ratios between 0.16 and 1.0). This enabled the critical factors controlling punch-through to be investigated parametrically due to the advantageous large testing area of the drum channel. These tests covered the range of geometries of practical interest to the jack-up industry, where punch-through failures are prone to occur during spudcan
installation. The trapped sand plug heights, which largely control the bearing capacity in the clay layer following punch-through, were measured from the post-testing samples. This set of tests provided additional evidence used to form the basis of the proposed sand plug to geometry relationship.

Following the medium dense sand centrifuge tests, the results were used in combination with very dense sand overlying clay test data reported in the literature. This was then used to recalibrate an existing failure-stress-dependent model, modified to predict the peak resistance whilst accounting for mobilisation induced embedment, covering a wide range of sand relative densities and sand layer height to diameter ratio, $H_s/D$. A power law to represent the variation of $D_F$ with $H_s/D$ was proposed, superseding the original bilinear relationships of Lee (2009) and Lee et al. (2013), which inaccurately reflected the spudcan peak resistance response for smaller $H_s/D$. All the input parameters can be obtainable from standard site investigation techniques. The design equations for the peak resistance were coded and executed in an Excel spreadsheet, allowing predictions and sensitivity analyses to be performed quickly. The calculated peak resistances now match both medium dense and very dense sand experimental results well. By incorporating the properties of the sand and considering the embedment depth at $q_{\text{peak}}$, the modified model showed improvement over the prediction methods advocated in the current offshore industry guideline ISO (2012).

8.2.2 Full profile prediction method for the prediction of punch-through

The full penetration resistance profile for foundations on sand overlying clay is of great importance in the evaluation of the severity of a potential punch-through failure and to shed light on the need for proper mitigation measures. A simplified design method for producing a full penetration resistance profile for spudcan foundations on sand
overlying clay has been proposed in Chapter 4. The method was established based on a combination of conceptual models and design equations, developed through the combination of physical modelling using a geotechnical centrifuge and numerical modelling using the CEL approach. The sand was modelled using the Mohr-Coulomb model, while the clay was modelled using a modified Tresca model to account for strain softening. The numerical method was used to simulate a series of spudcan penetration tests on medium dense sand overlying clay for a single 13° conical angle spudcan. The effects of the \( H_s/D \) ratio, the undrained shear strength of the clay at the sand-clay interface and strength gradient of the clay were explored in a complementary parametric study. A simple linear expression for the bearing capacity factor for the spudcan and underlying sand plug was summarised from the above analysis. This expression, combined with the general expression for the bearing capacity in the underlying clay layer and the modified failure stress dependent model for predicting peak resistance, formed a simplified method for the prediction of the full penetration resistance profile.

The stress level dependency and dilatancy response of sand were accounted for in the calculation of the peak penetration resistance in the upper sand layer, while the thickness of the trapped sand plug and the shear strength profile of clay were incorporated in the design equation to calculate the resistance in the underlying clay layer. This new method also provides estimates of the potential punch-through distance.

8.2.3 Half-footing test investigating the shape effect of a footing penetrating sand overlying clay

The analyses previously reported in the literature were mainly for conical spudcan geometries with 13° underside angle, except for few tests performed using a flat footing and a spudcan with ~10° underside angle. The effect of footing shape on the peak
resistance, the resistance in the clay and evolution of soil deformation mechanisms was investigated in Chapter 5. Photographs of soil displacements were taken in-flight and analysed using the PIV technique. Using the aforementioned failure stress dependent model, the peak resistance was assumed to be equal to that measured in the experiments, allowing the magnitudes of the distribution factor, $D_F$, to be back-calculated. The $D_F$ values were found to be similar for footings with conical angles between 7° and 21° for a given $H_s/D$. The assumption adopted in the failure stress dependent models of Lee et al. (2013) and of this thesis that the inclination angle of the trapped sand wedge is approximated as the dilation angle of the sand was validated by inspection of the normalised incremental deviatoric shear strain contours mobilised at $q_{\text{peak}}$. Based on all the testing measurements in this research and previous studies, a constant value of $0.9H_s$ was confirmed to be an adequate approximation of sand plug height in the underlying clay layer.

### 8.2.4 Generalised design equation for the bearing capacity factor

The aim of the numerical analyses in Chapter 6 was to back-calculate the bearing capacity factor when the footing and the sand plug underneath penetrated into the underlying clay layer, considering various footing shapes, relative density of sand and undrained shear strength of clay. The upper sand layer was modelled using a Modified Mohr-Coulomb model and the underlying clay was modelled as a modified Tresca model, both of which considered the softening behaviour of the soils. The analytical model for predicting peak resistance was validated for medium dense to very dense sands and conical footings of angle 0° to 21°. Based on a large database of the back-calculated bearing capacity factor from both physical and numerical analyses, a linear equation was summarised to estimate the bearing capacity factor of both the spudcan and conical footings (with conical angles varying between 0° and 21°) reasonably well.
The bearing capacity factor was practically independent of the relative density of the sand, the investigated footing shape and shear strength of clay. This combined with the expression for the sand plug height, forms the two components of the bearing capacity in the underlying clay layer: the bearing capacity for a weightless soil and buoyancy. The bearing capacity was expressed in a simple form, enabling easy and fast evaluation of the potential for and severity of punch-through failure when combined with the failure stress dependent model for predicting $q_{\text{peak}}$. By comparing the penetrating resistance profiles from the prediction method developed with available centrifuge testing and field data, the prediction method developed provided reasonable estimates of full profiles and punch-through distance.

**8.2.5 Evaluation of performance of full profile prediction methods for the prediction of punch-through**

Chapter 7 evaluated the performance of the existing full profile prediction methods, i.e., the Teh method, Lee et al. method and an interpretation of the ISO (2012) guideline, and compared these to that developed in this thesis. A total of 45 centrifuge tests in the literature and the 26 tests of this thesis formed a large database for the evaluations. Comparisons were made on the peak resistance $q_{\text{peak}}$, the bearing capacity factor $N_c$, and the punch-through distance $d_{\text{punch}}$ from each prediction method and that measured in the centrifuge tests. By comparing the accuracy, scatter and geometric skew of the predictions from each method, the Teh method, Lee et al. method and ISO method were found to be biased with $H_s/D$. The Teh method showed skew in the $q_{\text{peak}}$ and $N_c$ predictions with respect to $H_s/D$, which in turn resulted in the failure of the method to capture the punch-through potential for some cases. Although the Lee et al. method predicted $q_{\text{peak}}$ extremely well, poor estimation of the punch-through distance was
evident due to a certain amount of skew in the calculations of $N_c$ with respect to the $H/D$ ratio. Due to the adoption of the inappropriate simplified failure mechanisms and the effect of the sand plug on the bearing capacity factor in the underlying clay layer being neglected, the ISO method produced the worst predictions in all the methods. In contrast, the method proposed in this thesis provided largely unbiased predictions for the above key calculations and thus more reliable punch-through failure and punch-through distance. The constructed penetration resistance profiles, which captured the distinctive aspects of a typical spudcan penetration resistance profile, were comparable with those obtained experimentally. The majority of the predicted punch-through distances were within the range of ±20% of the ones from centrifuge tests, which added more confidence in its application for predicting the potential for and severity of punch-through failure.

8.3 RECOMMENDATION FOR FUTURE WORK

8.3.1 Increased flexibility of sand model

Large deformation finite element analysis is necessary to model continuous penetration of a spudcan on sand overlying clay. A Modified Mohr-Coulomb model for sand response from peak to post-peak was incorporated into the Coupled Eulerian-Lagrangian analyses. However, the majority of the predicted peak resistance were still somewhat lower by ~11.0% (on average) than those measured in the centrifuge tests. The Modified Mohr-Coulomb model may be updated or replaced with more advanced models in the future to reproduce the sand behaviour more faithfully.

8.3.2 Investigation of footing behaviour in multi-layered soil

In this study, a simplified method was proposed to predict the full penetration resistance profile of the spudcan or conical footing in sand overlying clay. The clay layer is
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featured with undrained shear strength increasing linearly with depth. There are many offshore regions where the soil is stratified with inter-bedded layers of softer or stiffer clay. For example, the V-H-M envelope and the stability of the sand plug for a soil stratigraphy of sand-clay-sand are concerned by industry. It is also worthwhile to extend the prediction method to multi-layered soils. More centrifuge model tests and parallel large deformation analyses are necessary to produce continuous penetration profiles to gain insight of the failure mechanism developed in the multi-layered soil. The full profile prediction methods for multi-layered soil may be established by combination of the different analytical approaches for sand, clay and sand overlying clay.

8.3.3 Improving the predictions between $q_{\text{peak}}$ and sand-clay interface

The proposed method connects $q_{\text{peak}}$ and the penetration resistance in the clay layer to construct the full profile and evaluate the potential and severity of punch-through failure. The penetration resistance profile between $q_{\text{peak}}$ and sand-clay interface is also the critical zone for determining the actual leg plunge when the effect of hull buoyancy during punch-through is incorporated. It is of most interest to practitioners for determining the final hull inclination after punch-through has occurred. Therefore, more investigations may be conducted to define the penetration response between $q_{\text{peak}}$ and sand-clay interface for further improvement of the prediction of punch-through failure.

8.3.4 Statistical interpretations of the full profile prediction method

All the full profile design equations in this thesis have been presented as if the parameters defining the equations could be determined with certainty. In reality, every variable used in the calculations is subjected to some margin of error. It is possible to address this problem statistically by assigning standard deviations or coefficients of variation to all the parameters (see Houlsby 2010 and Bienen et al. 2010). In this way, the uncertainties can be accounted for and quantified in a rational manner.
8.4 CONCLUSIONS

Safe installation of offshore jack-up spudcan in soil stratigraphy of sand overlying clay requires the complex challenge of accurately predicting punch-through failure prior to rig deployment. In this thesis, a series of physical and numerical modelling investigations were utilised to develop a full profile prediction method for spudcan penetration in sand overlying clay. With better predictive capabilities compared to the methods recommended in the current industry guideline and those published in the literature, the method proposed will be useful for industry to improve the prediction of jack-up behaviour under vertical loading on sand overlying clay stratigraphy.

8.5 REFERENCES


