LARGE PENETRATION OF SPUDCAN FOUNDATION IN MULTI-LAYERED CLAYS – NUMERICAL STUDY

by

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ABSTRACT

Since 1951 when spudcans were first introduced to the Offshore industry, mobile jack-up rigs have been extensively used for offshore drilling. Over the past decades, the potential geohazards during jack-up rigs’ installation processes have been researched by numerous engineers and scholars. The unexpected rapid leg penetration referred to as “punch-through failure” has been recognised and studied for decades. However, it is still a challenge to accurately predict the spudcan behaviour in multi-layered soils.

The objective of this research is to establish a soil-flow-mechanism-based design approach to predict the full profile of spudcan bearing response in a three-layer clay stratigraphy. The current design guidelines and recent research work present a “step-wise” approach, only providing estimated closed form bearing capacity solutions for key penetration depths (e.g. where punch-through failure is likely to happen), owing to the lack of solutions for continuous penetration response and the effect of soil layer deformation on foundation capacity.

To simulate the spudcan penetration in layered soils, an extensive Large Deformation Finite Element (LDFE) analyses were performed using Remeshing and Interpolation Technique with Small Strain model (RITSS). As per the stratified soil condition in Australia offshore fields according to previous study while also be able to capture the punch through failure the soil profiles in this study included: i) a soft uniform clay layer interbedded in a stiff uniform clay; ii) a stiff uniform clay layer interbedded in a soft uniform clay; iii) a stiff uniform clay layer interbedded in a normally consolidated (NC) clay. Numerical results were calibrated against centrifuge testing data, before the parametric studies were conducted.

A wide range of soil properties were selected based on the survey data of offshore deposits. The effects of multiple soil parameters, such as soil layer strength ratio and soil layer thickness ratio, on the spudcan penetration responses were studied, and the soil failure mechanisms were explored.

The predictive framework of the full profile of spudcan penetration in layered clays is proposed based on the extensive parametric study in this research. The LDFE/RITSS analyses have provided the link between the spudcan bearing response and the soil flow mechanisms during the continuous penetration of spudcan. Thus the predictive framework is mechanism based, which includes soil layer deformations and continuous soil movements.

Based on the soil failure models identified in the study, a series design formulas are proposed corresponding to the spudcan penetration depths. The proposed framework is tested with a large database of model test results. Therefore it can be used as a design tool for assessing “punch-through failure” risk when jack-ups with spudcan foundations are installed on the seabed with multilayered clays.
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## PUBLICATIONS

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CHAPTER 1 INTRODUCTION OF JACK-UP RIGS AND SPUDCAN FOUNDATIONS

1.1 BACKGROUND OF OFFSHORE OIL AND GAS INDUSTRY

Global demand for energy has risen inexorably in the last 150 years in step with industrial development and population growth. As developing countries like China and Middle East countries seek to fuel their rapid economic growth, hunger for energy is predicted to continue to rise in future. Take oil for example: the global demand grows steadily with a rate of 1.15%/y (Figure 1) from 2012 to the near future, according to the data from International Energy Agency (IEA).

![Global Oil Demand Chart](chart.png)

Figure 1 Global oil demand increases over 2012-18 (IEA, 2013)
Offshore drillings for oil off the coast were firstly performed around 1896 in the portion of the Summerland field extending under the Santa Barbara Channel in California (Lindstedt et al., 1991). Another offshore milestone was achieved in 1947, when Kerr-McGee Oil Industries drilled the first productive well beyond the sight of land, located 10.5 miles (i.e. 16.9 kms) off the Louisiana coast, though still in water depths of only about 5.5m (Pratt et al., 1997). Since then each new drilling project advanced the state of technology. In 1954 the first purpose-built Mobile Offshore Drilling Unit (MODU), named “Mr. Charlie”, was designed with pontoons on both sides to provide better stability. Later on, Shell Oil saw the need to have a more motion-free floating drilling platform in a stormier water field by adding an anchor system on a submersible, converting a submersible to what is now known as a semisubmersible. The first purpose-built drilling semi-submersible “Ocean Driller” was launched in 1963 and was still serving well until 1992 (Howe, 1986). As advances in technology opened up large swathes of the offshore to the possibility of drilling, the exploration of oil and gas has ventured into the fields with deeper water and harsher environment. In 1975 Shell Oil Company discovered the first deep-water field in Cognac (operating at a water depth of 300m below ocean surface). Although the water depth was relatively deep, the platform in that field was still designed based on steel fixed structures suitable for shallow waters. Throughout the 1970’s the number of jack-up rigs constructed increased rapidly. By the end of the decade, they represented half of the mobile unit fleet. Hull motion suppressors and stronger legs were introduced to improve the performance of legs in deeper water, providing greater strength and ability of weatherproof for harsher environments. Since 1980’s, the emphasis of drilling was focused on the final two frontiers, ice and ultradeep water. A number of jumbo jack-ups were built for use in
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harsh wind, wave and temperature conditions. Nowadays a variety of mobile offshore drilling rigs is available for service, including semi-submersible platform, drilling ships, FPSO (floating production, storage, and offloading system), etc. to cope with different environment and water depth, as shown in Figure 2.

![Types of Offshore Oil and Gas Structures](http://oceanexplorer.noaa.gov/)

**Figure 2** Types of Offshore Oil and Gas Structures: 1, 2) conventional fixed platforms; 3) compliant tower; 4, 5) vertically moored tension leg and mini-tension leg platform; 6) spar; 7,8) semi-submersibles; 9) floating production, storage, and offloading facility; 10) sub-sea completion and tie-back to host facility. (source: [http://oceanexplorer.noaa.gov/](http://oceanexplorer.noaa.gov/), 2008)

1.2 MODU AND JACK-UP UNITS

In the early 1950s, drilling engineers and facility designers were trying their best to find ways to drill in open water at depths greater than 5~6 m. A young U.S. Navy engineer named Alden James “Doc” Laborde came up with a plan to put the entire drilling operation on a transportable barge that could be floated to any location in water depth up to 12 m. This innovation was named “Mr. Charlie” (Figure 3), constructed in early 1952 and was firstly employed in 1954 on East Bay Field at the mouth of the Mississippi River for Shell Oil Company.
With “Mr. Charlie” working well in the field as a mobile offshore drill unit, a new concept in the form of a “jack-up” showed up (Howe, 1986). A jack-up rig is a self-elevating unit and a type of mobile platform that consists of a buoyant hull fitted with a number of movable legs, capable of raising its hull over the surface of the sea. Very rapidly it become the most popular mobile type in existence, for water depths suitable to the available leg lengths.

The forerunner of today's jack-ups was named “Scorpion”, designed by R. G. Le Tourneau and built by the Le Tourneau Company in 1956 (Veldman and Lagers, 1997). It had independent legs with spudcan foundations working in a water depth of 24 m. In the late 1990s, more effective module of jack-ups were designed and built to carry larger deck loads (≥ 3100 tons), and drill in deep water depth (≥ 120m) (source: Centre for Offshore Foundation Systems (COFS))
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http://petrowiki.org/ 2014). Although jack-ups were initially designed with 6 to 8 legs and a few with 4 legs, the vast majority of the units nowadays have 3 legs. Among the various types of mobile platforms commonly deployed in shallow and moderate water depths, the jack-up rig has been the most popular type in existence with 57% share of the worldwide offshore rigs (Figure 4). The total number of Jack-up 'Drilling' rigs available is approximately 445 at the end of 2014, and is reaching over 600 in 2017 (source: https://www.rigzone.com/data/, 2014, 2018).

![Worldwide utilization rate of offshore rigs](source: www.rigzone.com 10 November 2014)

A typical jack-up rig usually consists of a hull supported by three or four steel truss legs, with spudcan foundations at their feet. A picture of an example jack-up unit is shown in Figure 5. Each leg is fitted with a rack and pinion system to jack the legs up and down through the deck during installation, and to lift the jack-up platform clear of the water during operation.
Large inverted cone-shaped foundations, which are known as spudcans, are widely used to support each independent leg. Figure 6 shows that a spudcan foundation is roughly circular in top view, with a shallow conical underside (in the order of 15° to 30° to the horizontal) (ISO, 2012) and also a sharp protruding tip 1~2 m in length. Large jack-up spudcans can be in excess of 20 m in diameter, with their shapes varying with the manufacturers and jack-up units. A typical spudcan size found in industrial applications falls in a range of 10-20 m in diameter. Some other spudcan geometries easily found in applications are shown in Figure 7. The non-circular spudcan is normally approximated as a circular shape with an equivalent diameter during assessment processes.
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Figure 6 A barge loaded with a leg section and spudcan of the jack-up rig PRIME EXERTER

(Photo: Monique Davis-Mulder © source: DMNC)

Figure 7 Typical non-circular spudcan configurations (ISO, 2012)
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1.3 FOUNDATION HAZARDS OF SPUDCANS IN A MULTI-LAYERED SEABED

During the installation of a jack-up unit, “preloading” is required to drive the legs into the seabed before the hull is completely jacked out of the water. During this process, the major concern is unexpected sudden penetration of one of legs/foundations under preloading. The prediction of spudcan penetration is one of the challenges for offshore foundation engineers, especially in a seabed with a layered soil profile. When a strong soil layer is underlain by a soft soil layer, the foundation may experience a peak resistance during penetration through the strong layer, followed by a sudden penetration (i.e. ‘free fall’) without any additional loading. This sudden penetration is termed ‘punch-through’ failure (Hossain and Randolph, 2010, 2010, Gao et al., 2012), which is most likely to occur during the preloading phase during the installation process (Figure 8). The upper soil layer will provide a peak bearing capacity and if this value is exceeded during preloading, then a rapid penetration will occur for a certain distance until sufficient resistance from soil is gained at greater depth.
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A stratified soil formation is commonly found in the regions with active jack-up rig operations. Historical cases indicated that one third of jack-up accidents recorded before 2002 have been associated with foundation problems (shown in Figure 9) and punch-through failure has the highest rate in incident causes, representing 53% of all incidents (Dier et al., 2004). Take Southeast Asia operating area as an example: it was estimated in year 2002 that in the S. E. Asia operating area typically one or two major punch-through failures are occurring each year, together with about 10 to 12 unexpected rapid footing penetrations. (IADC, 2002)
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Figure 9 Case histories classified according to the cause of failure (Dier et al., 2004)

The punch-through failure at the foundation level can cause damage to the offshore structure that the foundation supports. For example, on 16th Nov 1996, all three of the Maersk Victory jack-up's legs got extensive damage due to the punch-through failure of one leg as seen in Figure 10 (Aust, 1997). The failures of the Noble David Tinsley in May 2009 with severe damage to the legs and rig, and the incident of the Sapphire Driller in October 2009 during a preloading test were also recently reported. Another recent example is the jack-up drilling rig Perro Negro 6 incident that happened near the mouth of the Congo river in July 2013 with one of the rig’s crew of 103 reported missing and six others injured.
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1.4 SITE ASSESSMENT STANDARD (ISO 19905-1, 2012)

In the late 1980s a Joint Industry Project managed by Noble Denton was funded to develop a jack-up assessment methodology. And in 1994, SNAME (The Society of Naval Architects and Marine Engineers) published "T&RB 5-5A Site Specific Assessment of Mobile Jack-Up Units" as the results of that work (Hoyle et al., 2006). The background to the project was disseminated at a seminar (Various, 1993) held at City University, London, in September 1993. There have been two revisions since the first edition. The first revision was issued in May 1997 and the second in January 2002. It has been widely used to assess the bearing capacities of spudcan foundations which undergo progressive penetration during preloading. During the years, the document has been continuously modified and updated. SNAME Recommendation was revised in 2008. However the treatment of punch through in layered soil is still largely based on the understanding from onshore practice, which inevitably has its limitations to offshore design. The bearing capacity formula has been derived empirically for surface foundations and does not account for spudcan roughness and shape (SNAME, 2008). For a spudcan penetrating into a layered soil system, particularly where a stiff layer

Figure 10 Maerak Victory punch-through failure(Aust, 1997)
overlaying a soft layer, it is a common understanding that a soil plug could be formed as shown in Figure 11.

![Figure 11: Process of the soil plug forming](image)

However, the effect of soil plug from the top strong soil layer is not considered in the assessment. The evaluation of punch-through potential for a spudcan penetrating into a strong over soft clay layer is computed according to Brown (Brown and Meyerhof, 1969), where the bearing capacity factor was determined based on experimental studies with circular footing on homogeneous clay. The framework to assess the spudcan bearing capacity profiles in multi-layer soil profiles doesn’t include soil layer deformation and the continuous change in soil flow mechanisms. The bearing capacity of a spudcan is analysed as a “wished-in-place” case, where the foundation is moving downwards without the disturbance of the soil layers.

Based on the decades practice of spudcan foundation assessment by the SNAME guidelines, in 2012, ISO 19905-1 as an ISO standard for the Site-Specific Assessment of Mobile Offshore Units was established. In this establishment, SNAME TR5-5A 1-12
Revision 3 was used as a starting point. Since then, the ISO 19905-1 standard has been subjected to periodical updates. However, to date, only one update was received in 2016 with corrections based on the findings from industry case studies and academic research results. In the latest version of ISO-19905-1, the shape and depth factors from Hansen’s study (Noble, 2009) were replaced by those proposed by Skempton (Petroleum, 2009) due to a long-established reputation and sufficient accuracy the method can provide. An alternative calculation method which provides tables of bearing capacity factors (Houlsby and Martin, 2003) for conical foundations with a series of embedment ratios is described in ISO 19905-1. Those factors take direct account of cone shapes, embedment depths, foundation roughness and soil properties. Also most significantly, a rational method to predict the depth at which a backflow failure mechanism becomes preferential for spudcan penetration (Hossain and Randolph, 2009) is introduced in ISO 19905-1. The cause of the soil backflow is proved to be a preferential local soil flow mechanism due to spudcan penetration, rather than the instability of the open cavity described in SNAME 5-5A. Assessment methods for spudcan penetrating into layered soils are presented in ISO 19905-1. Although a series of updated soil failure models are illustrated in ISO 19905-1, such as punch-through with plug trapped underneath or squeezing, there are no corresponding equations are provided for a continuous penetration resistance profile. Therefore it is imperative to further investigate spudcan on layered soils and to provide a more advanced analysis approach for design by offshore design engineers.

1.5 EVALUATION OF SPUDCAN VERTICAL BEARING CAPACITY

- The conventional procedure of calculating vertical bearing capacity of spudcan foundation during preloading is provided by current mainstream guideline (ISO, 1-13)
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2012) as shown in Figure 12. The gross ultimate vertical bearing capacity is calculated based on the ultimate bearing capacity formulation for shallow foundation with the best estimate of the soil strength parameter at every prescribed embedment.

Figure 12  The conventional procedure for the assessment of spudcan load/penetration behaviour

(ISO, 2012)

Due to the limitations in the current assessment guidelines discussed above, especially where the seabed contains multi-layer soil systems, some publications appeared in recent years in order to provide more advanced assessment to deal with foundation potential punch-through failures. However, these recently developed assessment methods have not been considered as primary recommendations in the mainstream assessment guidelines, since they have yet to become fully established and they need to be examined and tested through practices.
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The relevant research found in the literature are listed below categorised by soil layer profiles.

1) Spudcan in stiff clay over soft clay: studies were conducted to investigate the sensitivity of soil parameters and foundation roughness on punch-through failure mechanism. (Liu et al., 2005, Hossain and Randolph, 2010, 2010, Gao et al., 2012). It was found that severe punch-through was associated with a soil plug formed by the upper layer soil moving downward into the soft soil layer. The formation of the soil plug involves punching shear, as clear shear planes are formed around the soil plug in the shape of a truncated cone from the strong upper layer underneath the foundation. An assessment approach (Hossain and Randolph, 2009) based on soil flow mechanism was proposed to assess the spudcan penetration resistance profiles through stiff-over-soft clays. Both strain rate and strain softening of clays were considered in the study.

2) Sand over clay: The punching shear method (Hanna and Meyerhof, 1980) is recommended in the current assessment guideline (ISO 2012). The key parameter is the punching shear coefficient $K_s$, which depends on the strengths of both sand layer and clay layer. However it is still a wished-in-place assessment method without any consideration of the soil layer distortion due to the foundation penetration. (Teh et al., 2009) and (Lee, 2009) proposed alternative methods for the peak capacity based on the soil failure mechanisms observed in the centrifuge tests, where the sand plug was pushed into bottom clay layer with an outer angle reflecting the dilation of the top sand as shown in Figure 13.
Figure 13 Sand plug into clay layer with an outer angle reflecting sand dilation (Lee, 2009)

3) Multi-layer system: spudcan on the seabed with stratified soils (three layers or more), which were commonly encountered in practice, was also studied (Pratt et al., 1997, Hossain and Randolph, 2011, Hossain, 2014). Spudcan penetration in multi-layer soils up to 6 layers were tested in centrifuge. The results showed that soft layer can also be trapped as a soil plug beneath the foundation and carried downwards instead of being squeezed away. However, so far, there are limited number of detailed algorithms available for assessing spudcan penetration in a multi-layer system. (Xie et al., 2010) proposed an algorithm which is similar to the current assessment guideline (bottom-up method), assuming each layer is independent and the interaction between layers is neglected.

1.6 OBJECTIVE AND SCOPE OF STUDY

The current assessment guideline for spudcan penetrating into multilayered soil is based on the following major assumptions, i) for soft-over-stiff clays, the squeezing
mechanism is assumed occurring in the top soft soil only and the bottom stiff clay is acting as a rigid boundary; ii) for stiff-over-soft clays, the top stiff layer can bend downwards to the soft bottom layer as punch-through failures. However, no soil layer deformation is taken into account in foundation failure assessments.

Based on the identification of the limitations of the current assessment codes, this study took initiative to explore spudcan performance on multi-layered clays through large deformation finite element (LDFE) analysis. Three prevalent configurations of soil profiles were adopted as: (a) uniform clays having a soft layer sandwiched in the middle, to understand the relationship between spudcan bearing capacity curve and soil flow mechanism, (b) a stiff layer (uniform clay/ sand) interbedded in a soft uniform clay, to understand the relationship between spudcan bearing capacity curve and soil flow mechanism, (c) clays with linearly increasing strength with a stiff layer sandwiched in the middle, to quantify the effects on spudcan bearing capacity introduced by different soil flow mechanisms.

The key aspect of this study is to investigate the mechanisms of soil flow generated with the progress of spudcan towards deep penetration. The continuous penetration large deformation finite element (LDFE) analyses can provide penetration resistance associated with corresponding soil flow for a range of soil parameters including soil undrain strength, shear strength gradient and soil geometry (i.e. normalised layer thickness relative to foundation diameter) which cover a wide range found in practice. The main focus is to develop a mechanism based approach to assess spudcan penetration in these seabed conditions with multi-layered soils. The outline of this thesis is displayed below.
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1.7 THESIS OUTLINE

Chapter 2 reviews the literature related to the foundation bearing capacity in single and layered clays. The development in bearing capacity of shallow foundation and deep penetrated foundation on single layered clay with focus on the conical shape and roughness effects, soil failure mechanism and bearing capacity formulations were summarized. For bearing capacity in layered clays, the literature review includes existing experimental observations, progresses made in numerical studies, and also updated assessment methods. Finally, the assessment recommendations by ISO 2012 to assess the spudcan bearing capacity are discussed.

Chapter 3 reviews the development in LDFE methods that were employed in similar study areas. Technical details of the current LDFE analyses which were used in this thesis are also provided.

Chapter 4 presents the studies of spudcan foundations in a seabed with a soft middle layer embedded into stiff uniform clays, identifying the soil flow mechanisms and the sensitivity of each soil parameters.

Chapter 5 presents the studies of spudcan foundations in soil with a strong middle layer embedded into soft uniform clays, identifying the soil flow mechanisms and the sensitivity of each soil parameters.

Chapter 6 presents the studies of spudcan foundations in soil with a strong middle layer embedded into normally-consolidated clays on spudcan penetration, identifying the soil flow mechanisms and the sensibility of each soil parameters.
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Chapter 7 accounts for soil layer distortion. Improved formulations have been proposed to obtain the entire spudcan penetration profiles, showing encouraging agreement between the predicted bearing capacities and LDFA analyses results.

Chapter 8 finally culminates the dissertation by providing a summary of findings and conclusions.

1.8 REFERENCES


1-19
Chapter 1: Introduction of jack-up rigs and spudcan foundations

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2.1. INTRODUCTION

During the life cycle of offshore structures the seabed has to be able to withstand the weight of these structures and the applied loads to ensure the operations are conducted safely. Comparing with onshore structures, there are a few more challenges when designing offshore foundations, including more expensive site investigations, unusual soil conditions, different construction and installation procedures, significant higher costs and severe consequences in case of failure and so on. For many years, offshore designs were dominated by case histories and past experiences. However the soil conditions at each offshore location have been proved radically different. The unexpected sudden penetrations of spudcan foundations during their installation phase are a major risk to the stability of jack-up platforms. The sudden large displacement of one or more jack-up legs could result in damage to the structure of the jack-up, construction down time, or sometimes fatality. Thus, it is important to predict the spudcan penetration behaviour accurately in the design phase. However, it is proven to be more challenging to do so for complex soil profiles encountered offshore. The study by (Osborne et al., 2009) shows that, due to geological processes, highly layered soil profiles are prevalent at seabed in areas such as the Sunda Shelf, offshore Malaysia offshore Thailand, offshore India, offshore Australia and in the Arabian Gulf. Therefore a study to assess the risk for spudcan sudden penetration into multi-layered seabeds is eagerly urged, which can form the basis to establish design guidelines.
2.2. FOUNDATION ON SINGLE-LAYER SOIL

The bearing capacity of a vertically loaded, shallow strip foundation on a homogeneous soil is traditionally calculated using the conventional bearing capacity theory of (Terzaghi, 1943), which assumes that the soil is rigid-perfectly plastic with the strength characterised by a cohesion, $c$, a friction angle, $\phi'$, and an effective unit weight, $\gamma'$. Following (Prandtl, 1921), the soil failure mechanism consists of a rigid triangular wedge beneath the strip foundation, a radial shear plane and an emerging passive wedge, as shown in Figure 1

![Figure 1 Failure mechanism of classical bearing capacity for a strip foundation(Terzaghi, 1943)](image)

(Terzaghi, 1943) proposed that the individual contributions to the foundation bearing capacity from $c$, $\phi'$ and $\gamma'$ can be superimposed. Hence the foundation bearing capacity can be expressed as:

$$q_{ult} = N_c c + N_q q_0 + N_\gamma \frac{\gamma B}{2}$$  \hspace{1cm} \text{Equation 1}

where $q_{ult}$ is the ultimate (average) vertical pressure, $q_0$ the effective surcharge at foundation level and $B$ the foundation width. The parameters $N_c$, $N_q$ and $N_\gamma$ are the
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bearing capacity factors for soil cohesion, surcharge and soil unit weight effect respectively.

The exact solution for bearing capacity factor $N_c$, which is the focus in this thesis for spudcan, was obtained using plasticity theory (Prandtl, 1921), where a shallow strip foundation was placed on the surface of a weightless soil with constant undrained shear strength, $s_u$ (or cohesion, $c$). The exact expression of the $N_c$ factor is provided in Table 1:

<table>
<thead>
<tr>
<th>$\phi'$</th>
<th>$N_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi' = 0$</td>
<td>$2 + \pi$</td>
</tr>
</tbody>
</table>

Mohr-Coulomb soil model with values of $c$ and $\phi'$. $N_c = (N_q - 1) \cot \phi'$

Table 1 The exact expression of the $N_c$ factor (Prandtl, 1921)

And the values of the parameters $N_c$, $N_q$ and $N_\gamma$ for foundation on single layered soil only depend on the friction angle $\phi'$ shown in Figure 2.
Recently, there are more accurate forms of $N_r$ published by (Martin, 2005, Smith, 2005). However in this research the top layer soil is always clay under undrained conditions ($c = s_u$, $\phi' = 0$ for total stress analysis). Therefore the third term in equation Equation 1 is taken as zero.

Equation Equation 1 was intended for the bearing capacity of strip foundations at shallow depth. Terzaghi (Terzaghi, 1943) indicated that when the embedment depth was less than the foundation width, the shear resistance above the foundation base could be ignored. In order to assess the bearing capacity for foundations with various shapes and under different embedment depths, the classical bearing capacity equation has been extended in the form of:

$$N_q = e^{\pi \tan \phi' \tan \left( \frac{\pi}{4} + \frac{\phi'}{2} \right)} \quad \text{(Shield 1954)}$$

$$N_c = (N_q - 1) \cot \phi' \quad \text{(Prandtl 1921)}$$

$$N_r = 2(N_q + 1) \tan \phi' \quad \text{(Vesic 1973)}$$


\[ q_{\text{ult}} = (d_c s_c) N_c c + (d_q s_q) N_q q_0 + (d_\gamma s_\gamma) N_\gamma \frac{yB}{2} \]  

**Equation 2**

In this equation, dimensionless shape factors of \(s_c, s_q\) and \(s_\gamma\) have been introduced as coefficients to each bearing capacity term to take into account the foundation shapes of rectangular, square and circular. For circular foundations, \(B\) is the diameter of the foundation. Since this research focuses on circular foundations, notation \(D\) (implying diameter) is used in the thesis.

For this project, the focus is for clay under undrained conditions (\(c = s_u, \phi' = 0\) for total stress analysis). Thus Equation 2 can be simplified as below:

\[ q_{\text{net}} = N_c s_u \]  

**Equation 3**

Where \(q_{\text{net}}\) is the bearing pressure on the foundation and \(s_u\) is the undrained shear strength of soil in this study, where the shape and embedment depth effects are included in the factor \(N_c\) for spudcan. The net pressure \((q_{\text{net}})\) is calculated as:

\[ q_{\text{net}} = \frac{F_{\text{net}}}{A} = \frac{F_{\text{total}} - F_{\text{buoyancy}}}{A} \]  

**Equation 4**

In which \(F_{\text{total}}\) is the total reaction force acting on the offshore foundations in FE analysis, \(F_{\text{buoyancy}}\) is the spudcan buoyancy in soil and \(A\) is the projected area of the foundation, i.e. \(A = \pi D^2 / 4\), where \(D\) is the foundation diameter. Thus, the bearing capacity factor, \(N_c\), represents the bearing response of the foundations, which can be used in foundation design.

### 2.3. FOUNDATION ON DOUBLE LAYERED SOILS

In the past, many research on punch-through failure of offshore foundations studied spudcans or flat footings on double layered soils, especially strong over weak soils.
The bearing capacity of surface foundation usually depends on the strength ratio of soft layer to the strong layer $\frac{S_{\text{soft}}}{S_{\text{strong}}}$ and the thickness ratio of the top layer to the footing size $H_t/D$. Research by (Hossain and Randolph, 2010, 2010) have proved that the depth of foundation embedment ($d$) and the clay strength gradient ($k$) are also factors that affect the foundation penetration response. In this thesis, surface foundations on homogeneous ($k = 0$) and non- homogeneous ($k > 0$) layered soils are studied.

For a soil profile of weak over strong clay, bearing capacity factors were given by Vesic (Vesic, 1973), and these factors were obtained by interpolation between rigorous solutions. The capacity factors were given in the form of charts in the text entitled “Foundation Engineering Handbook” (Winterkorn and Fang, 1975).

For a soil profile of strong over weak clay, which is more prone to punch-through failures, Brown and Meyerhof (Brown and Meyerhof, 1969) did a study based on model tests, which were confined to surface loadings, using footings with rough bases. They suggested that the analysis assuming simple shear punching around the footing perimeter would be appropriate, and in this case the bearing capacity factors were given by the following equation:

$$N_c = 3.0 \left( \frac{H_t}{2D} \right) + 6.05 \left( \frac{S_{\text{soft}}}{S_{\text{strong}}} \right)$$  \hspace{1cm} \text{Equation 5}

For the circular footing on a single uniform clay layer, Wang and Carter (2002) studied bearing capacity of circular footings based on numerical simulations. They assumed the footing base and sides were perfectly smooth. According to their results the bearing capacity factor $N_c = 5.69$ was obtained. This result is about 6% lower than that for a
footing with a rough base (i.e. $N_c = 6.05$ by (Houlsby and Wroth, 1983). Wang and Carter (2002) also found that for surface footings on layered soils the bearing capacity factors for smooth footings were also lower than those for rough footings by 6%.

Wang and Carter (2002) have also computed the bearing capacity factors of surface smooth footings on two-layer clays using small strain FE analysis (see Figure 3). The current small strain FE results are compared with the solutions of rough footings by Brown and Meyerhof (1969). To make the comparison for smooth footings only, the bearing capacity factors given by Equation 5 for rough footings have been reduced by 6.3% to represent the bearing capacity factors for smooth footings as shown in Figure 3. It can be seen that the FE results agree well with the analytical solutions for smooth footings in relatively uniform clays (i.e. $c_2/c_1 = s_{usoft}/s_{ustrong} \geq 0.8$ for $H/D = 0.5$ and $c_2/c_1 \geq 0.5$ for $H/D = 1$). However, for distinctive two-layer soils (i.e. $c_2/c_1 < 0.8$ for $H/D = 0.5$ and $c_2/c_1 < 0.5$ for $H/D = 1$), the FE results are apparently higher than the solutions by Brown and Meyerhof (1969). The difference may be due to that, in small strain FE analysis, the footings were confined to the soil surface, and in model tests, the footings were displaced closer to the soft clay layer underneath when they reached their capacities.
Liu et al (2005) did similar studies on spudcan foundations on layered clays using FE analysis. Similar conclusions were reached to the ones by Wang and Carter (2002) as stated above. Figure 4 displays the FE results by Liu et al (2005) that agree well with
equation Equation 5 at relatively uniform soils \( (c_1/c_2 \geq 0.8) \). The FE results are above the analytical solutions for distinctive layered soils when \( c_1/c_2 < 0.8 \) at \( H/D = 0.4 \). It can also be seen that the soil strength ratio at critical point (at which the uniform clay and layered clay behaviours are distinguished) can change with different top soil layer thickness. For \( H/D = 0.8 \), the uniform soil behaviours can be reached when \( c_1/c_2 \geq 0.5 \). The layered soil behaviours can only be observed when \( c_1/c_2 < 0.5 \). When soil strength ratio is less than the critical one, the roughness of the spudcan has no influence on its bearing response. The details of the flow mechanism study for small strain analysis can be found in (Mehryar and Hu, 2004). They showed that apparent a soil punching shear failure in the top stiff layer was dominant and extended to the lower soft layer. Hence the top stiff soil beneath the spudcan moved together with the spudcan as one entity, thus the spudcan roughness had no effect on its capacity.

![Graph showing relationship between \( N_c \) and \( c_1/c_2 \) for different values of \( H/D \).](image)

(a) \( \frac{H}{D} = 0.4 \)

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Figure 4 Bearing capacity factors of surface spudcan from small strain analysis (Liu et al., 2005)

For LDFE analysis, the bearing response of rigid circular footing penetrating into double layered clays (stiff over soft) has been studied by Wang and Carter (2002) using AFENA (Carter and Balaam, 2000) numerical software. In their conclusions, for a circular footing on a double layered soil with $c_2/c_1$ ($s_{soft}/s_{strong}$) = 0.1 to 2/3, the load-displacement curves indicated a trend of approaching the ultimate capacity of a footing deeply embedded in the lower soft clay layer. However, for cases of $c_2/c_1$ ($s_{soft}/s_{strong}$) = 0.8 to 1, the ultimate bearing capacity is higher than the analytical value for a footing buried in the weaker lower layer. The circular footing in their study was set up with full vertical shaft, where soil back flow was prevented. This footing setup is different from the spudcan foundation where, during continuous spudcan penetration, soil back flow onto the top of the spudcan is inevitable. The soil back flow can play an important role in the foundation bearing response (Hossain et al., 2005).

In the study by Liu et al (2005), spudcan penetration into a double layered clay was investigated numerically. The soil profile has been set up as a strong soil layer 2-10
overlying a soft soil layer, with the soil strength ratio \( c_2/c_1 \) (\( s_{\text{usoft}}/s_{\text{ustrong}} \)) varied from 0.1 to 1.0. The diameter of spudcan foundation was kept as \( D = 14 \) m. The soil back flow onto the top of the spudcan was observed.

Figure 5 shows some LDFE results by Liu et al. (2005). During large penetration of a spudcan into the layered soils, the bearing capacity of the spudcan responds to the top soil layer first, thus the bearing capacity increases with penetration. When the spudcan moves closer to the soil layer interface, a reduction in the bearing capacity occurs, showing the influence of the bottom soft layer. The peak point of bearing capacity in the top soil layer means the starting point of capacity reduction with further penetration. This also indicates the potential for punch-through failure. The penetration depth with spudcan peak capacity or the potential for punch-through failure is affected by (a) the relative thickness of the top soil layer of \( H_t/D \); (b) the top layer soil strength \( s_{\text{ustrong}} \) (i.e. cavity formation above the spudcan in the top layer); and (c) the soil layer strength ratio \( s_{\text{usoft}}/s_{\text{ustrong}} \) (i.e. spudcan capacity reduction after peak). It is also found that the soil layer strength ratio of \( s_{\text{usoft}}/s_{\text{ustrong}} \) has little effect on the potential punch-through failure depth. However, the capacity reduction rate increases with increasing strength ratio of \( s_{\text{usoft}}/s_{\text{ustrong}} \), where the strength and thickness of the top soil layer are constant and the cavity depth is much smaller than the thickness of the top layer. When the top soil layer strength is very high and the cavity depth is larger than the thickness of the top layer, the strength ratio of soil layers has an important effect on the potential punch-through failure depth.
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Figure 5 Spudcan penetration response for $H_t/D = 1$ with unit soil weight 7 kN/m$^3$ (Liu et al., 2005)

Similar conclusion came from Brennan et al. (2006) and Hossain et al. (2010) and (2010). In these studies, a wide range of $H_t/D$ and $s_{usoft}/s_{ustrong}$ combinations were investigated for circular footings/spudcan foundations on double layered soils by both physical...
The previous studies found that the strength ratio of double layered soils (i.e. $S_{\text{soft}}/S_{\text{strong}}$) played a significant role on soil failure mechanisms. For $S_{\text{soft}}/S_{\text{strong}} \geq 0.6$, failure occurred mainly through a combination of general shear (i.e. classical failure mechanism with surface heave in Figure 1) and partial punching shear mechanisms. For $S_{\text{soft}}/S_{\text{strong}} < 0.6$ soil displacements were predominantly directed vertically downwards (i.e. punching shear failure) with minimal lateral movements. The shear plane extended more deeply into the lower soft layer, as the strength ratio decreased (Figure 6). It was also found that the roughness of the spudcan base have little influence on the penetration response as shown in Figure 7 when punching shear failure occurred in the top strong soil layer.

(a-1) Comparison of soil flow mechanisms between centrifuge test and LDFE analysis

(b-1) Critical punch-through mechanism for spudcan penetration in stiff-over-soft clay

$S_{\text{soft}}/S_{\text{strong}} = 0.62, S_{\text{sub}}/\gamma' D = 0.28, H_{\text{top}}/D = 1$: $d/D = 0.38$

$S_{\text{soft}}/S_{\text{strong}} = 0.2, S_{\text{sub}}/\gamma' D = 0.36, t/D = 1$: $d/D < H_{\text{top}}/D$
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(a-2) Comparison of soil flow mechanisms between centrifuge test and LDFE analysis

\( \frac{s_{\text{soft}}}{s_{\text{strong}}} = 0.62, \frac{s_{\text{ubs}}}{\gamma' D} = 0.28, H_{\text{top}}/D = 1 \): 
\[ \frac{d}{D} = 1.15 \]

(b-2) Critical punch-through mechanism for spudcan penetration in stiff-over-soft clay

\( \frac{s_{\text{soft}}}{s_{\text{strong}}} = 0.2, \frac{s_{\text{ubs}}}{\gamma' D} = 0.36, t/D = 1 \): 
\[ \frac{d}{D} \geq \frac{H_{\text{top}}}{D} \]

Figure 6 Punch-through failure mechanisms shown in (Hossain and Randolph, 2010)

(a) Effect of strength ratio \( \frac{s_{\text{soft}}}{s_{\text{strong}}} \) on bearing response \( H_{\text{top}}/D = 1, \frac{s_{\text{ubs}}}{\gamma' D} = 0.36 \)

(b) Effect of thickness ratio \( \frac{H_{\text{top}}}{D} \) on bearing response \( \frac{s_{\text{soft}}}{s_{\text{strong}}} = 0.2, \frac{s_{\text{ubs}}}{\gamma' D} = 0.60 \)

Figure 7 the penetration response comparison between rough base and smooth base (Hossain and Randolph, 2010)

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For double layered clay soil profiles with stiff over soft clays, the bottom soft layer thickness was assumed to be infinite. However when the soft clay thickness becomes limited, soil profile turns to three layered soils, where the bottom soil layer is stronger than the 2\textsuperscript{nd} soft layer. In this case, multi-layered soil profiles need to be considered for foundation penetration responses.

2.4. FOUNDATION ON MULTI-LAYERED SOILS

In recent years, highly layered soil profiles are prevalent in several areas with current offshore explorations (Osborne et al., 2009). These areas include the Sunda Shelf, offshore Malaysia and offshore Thailand and more recent in the North Sea, offshore India, offshore Australia and in the Arabian Gulf. A case study on soil profiles at Yolla showed that several stiffer calcareous layers were embedded at various depths (Young et al., 1984, Hunt and Marsh, 2004). Although there are suggestions in the design guideline (ISO, 2012) to deal with multi-layered soils, the suggestions are mainly based on existing solutions of foundation responses on single and double layered soils. These solutions are then extended their applications in stratified sediments with more than two layers. This extension has made the design of foundations in multi-layered soils challenging, especially for spudcan foundations with large penetration potentials during installation and operation phases. Centrifuge modelling of spudcan foundations penetrating through 3-6 soil layers of clay with interbedded stronger clay, silica sand and carbonate sand layers was reported by (Hossain, 2014). This study provided some insights into the layered soil behaviour when spudcan foundations penetrated in stratified sediments comprising layers of different drainage conditions and mineralogy. The soil deformations beneath spudcan were found to vary significantly among the range of soil stratigraphies studied. It was found that the soil shear plane developed in
the top layer could be extended into 3-4 underlying layers when the layers were relatively thin (i.e. \( t/D < 0.6 \)).

**Figure 8** Soil failure patterns when spudcan penetrating into multi-layered clay (top three layers) with softer soil on the top (Hossain, 2014)

Following on the centrifuge tests by Hossain (2014), three-dimensional large deformation finite-element (3D-LDFE) of spudcan with deep penetration into three-layer clays was conducted using the coupled Eulerian-Lagrangian (CEL) approach in Centre for Offshore Foundation Systems (COFS)
ABAQUS/Explicit software (Zheng et al., 2015). In this study, two stratifications, including uniform stiff-soft-stiff and non-uniform clay with an interbedded stiff clay layer, were considered. The effects of normalized soil properties (eg. Strength Ratio, Normalized Strengths etc.) and layer geometries on the foundation bearing response and soil flow mechanisms were discussed in terms of punch-through, rapid leg run and squeezing. It was found that, when the foundation was going through a soft layer, the foundation penetration resistance increased dramatically at certain distance approaching the stiff layer ahead. This distance was defined as limiting squeezing depth (Zheng et al., 2014). This limiting squeezing depth is a function of various factors such as the strength ratio between the two layers and soil stress to strength ratio, etc..

Moreover, when there was a soil plug trapped at the base of the advancing spudcan from the stiff layer above the current soft layer, the bottom stiff layer could be sensed earlier than a ‘clean’ spudcan (see Figure 9), and hence the corresponding limiting squeezing depth could be increased as the plug behaved as part of the foundation.

![Figure 9 early squeezing caused by trapped soil plug (Zheng et al., 2014)](image-url)
However, the work by (Zheng et al., 2014) was mainly focused on the cases with potential punch-through failures and thus the parameters were limited to the range where punch-through failure could be observed.

In this thesis, extensive parametric studies have been conducted covering a broader range of parameters and layer configurations for multi-layer soils. Comparing with the previous work reviewed above, more detailed studies were performed on soil flow mechanisms during spudcan penetration through multi-layered soils, which could help to establish a more generic failure mechanism-based design approach.

### 2.5. METHODS OF CALCULATING BEARING CAPACITY ON MULTI-LAYER SOILS

#### 2.5.1. ASSUMPTIONS

Methods for calculating the bearing capacity of multi-layer soils range from averaging the strength parameters (Bowles, 1988) and using limit equilibrium considerations (Meyerhof, 1974) to a more rigorous limit analysis approach (Florkiewicz, 1989). Semi-empirical approaches have also been proposed based on experimental studies (Meyerhof and Hanna, 1978). However, almost all of these studies were limited to footings resting on the surface of the soil and were based on the assumption that the displacement of the footing prior to attaining the ultimate load was very small. In some cases, such as those where the underlying soil is very soft, the footing will experience significant settlement, and sometimes even penetrate through the top layer into the lower layer. In these cases, the small displacement assumption is no longer appropriate, and the large displacement of the foundation has to be taken into account in design.
2.5.2. ISO

ISO 19905-1 is an ISO standard for the Site-Specific Assessment of Mobile Offshore Units. Since its establishment in 1997, this standard has been constantly revised and updated over time based on the findings from industry case studies, and moreover based on the academic research results throughout the recent 20 years.

ISO-19905-1 has been used widely to assess the bearing capacities of spudcan foundations which undergo progressive penetration during preloading. In this standard, the ultimate vertical bearing capacity of the spudcan is computed using closed form bearing capacity solutions for various depths below sea floor. During decades of development, for spudcan penetrating into clay seabed, the depth effects on its capacity has been well captured by the embedment depth factors proposed by Skempton (Skempton, 1951) method has a long-established reputation in providing sufficient accuracy on the solutions. For circular foundations with a conical base, the tables of bearing capacity factors by (Houlsby and Martin, 2003) with different embedment ratios are also adopted in ISO 19905-1. Those factors have taken many factors into account, such as apex angle of the conical base, embedment depth, foundation roughness and soil properties. The capacity prediction methods for spudcan penetrating into layered soils however are still lack of detailed research. In the latest revision of the standard (ISO 19905-1) a series of updated soil failure mechanisms are proposed. However, it is also recommended that the soil failure mechanism needs to be selected carefully when predicting the foundation capacity for a spudcan on layered soils with a large penetration potential, such as squeezing failure mechanism in a soft over strong soil and punch-through failure mechanism in a strong over soft soil. These methods are limited to predict foundation capacities in discrete embedment depths. A prediction model to
provide a full spudcan penetration response penetrating through multi-layered soils is still not available.

**2.5.3. RECENT RESEARCH**

(Zheng et al., 2015) proposed a mechanism-based design approach for assessing spudcan penetration in soft non-uniform clay with an interbedded stiff layer. The proposed approach which can be taken as a “top-down” approach is developed on the basis of the results from a parametric study through large deformation finite-element analyses and validated against centrifuge test data. In this study, a group of design formulas are proposed for assessing the limiting cavity depth and soil plug height. The framework of this method can be summarised as:

1. Determine input parameters: representative values of the soil properties, layer thicknesses and spudcan geometries;

2. Plot the gross penetration resistance profile in the 1st layer according to an improved punch-through criterion. In this proposed criterion, variation of soil plug thickness during penetration is taken into consideration including soil properties and layer thicknesses effects;

3. Estimate the depth of turning point in the spudcan resistance profile where punch-through failure occurs and the depth of soil backflow, that is the same as the average limiting cavity depth, and then calculate the gross bearing capacity at these depths;

4. Evaluate the depth of turning point in the spudcan resistance profile where soil plug thickness attains to minimum. And then plot the gross penetration resistance at this point.
5. Connect all the turning points, estimate limiting cavity depth $H_{cav}$, and then update the gross bearing capacity profile to ultimate bearing capacity profile.

Comparing with other recent studies on spudcan penetrating into multi-layer clay (eg: (Kellezi and Stromann, 2003, Xie et al., 2010, Hossain, 2014), the work by Zheng et al. (2015) was the first one to include the effect of soil plug thickness variation during spudcan penetration. However this proposed approach can only apply to a certain range of soil parameters (eg. $S_{soft} / S_{strong} = 1.3~2.5$) for three-layer soil profiles with a strong layer interbedded in a soft layer.

2.6. **REMARKS**

A brief review of research related to the bearing capacity response of offshore foundations on clays has been presented above. It is shown that, since the introduction of Terzaghi's bearing capacity theory in 1943, many theoretical advances have been made in better predictions of the foundation bearing capacity on single and layered soils .

During the early period, adjustment factors to the classical bearing capacity equation were adopted to allow for variations from the original assumptions. However, more recently, this approach has gradually been replaced with more rigorous solutions from numerical analysis using modern computational technology. One thing worth of mentioning is that, during recent development in this research area, numerical methods have been proven working well with bearing capacity calculation especially when large deformation analysis is involved. Since then attention has been focused on exploring soil continuous deformation effects on foundation bearing behaviours, which has been proven important in assessing the foundation responses, especially in layered soils.
Recent studies and prediction methods for spudcan foundation on multi-layered soils have also been reviewed. While improvement has been made to obtain reasonable predictions on spudcan bearing capacities during penetration process, there is still a need to develop a design method that can provide full resistance profiles of spudcan penetration in layered soils; and that balances accuracy and practicality for the offshore engineering community. The method should be easy to use in practice, and with unambiguous parameters that can be derived from standard offshore site investigations.

2.7. REFERENCE

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CHAPTER 3 LARGE DEFORMATION FINITE ELEMENT METHODS AND RITSS

3.1. INTRODUCTION

Deep penetration of offshore foundations into multilayered soil has always been a challenging geotechnical research area, due to large deformation of geo-materials and soil layer rupture. The study in this thesis is regarding the spudcan foundations under pre-loading during jackup rig installation, where the spudcan foundation could undergo continuous penetration, with its final penetration depth more than two to three spudcan diameters into the seabed (ISO19905-1, 2012). Unlike most onshore and offshore foundations, where only surface foundation or foundation with shallow embedment needs to be designed, the existing design formulas for the shallow foundations can’t be used for the design of foundations with large penetrations. Due to the continuous penetration of foundation, involving complex and ever-changing soil flow mechanisms and soil layer distortion with its penetration depth, the soil-foundation interactions need to be investigated to develop proper design formulas to predict foundation capacities during its installation and operation.

In this study, the large deformation finite element (LDFE) analysis was conducted using Remeshing and Interpolation Technique with Small Strain (RITSS) model (Hu and Randolph, 1998, 1998). In LDFE/RITSS analysis, a series of incremental small strain analysis using the AFENA software package (Carter and Balaam, 2000) is combined.
with fully automatic remeshing of the entire soil domain, followed by interpolation of all field variables, such as stresses and material properties, from the old mesh to the new mesh. The existing framework of the LDFE/RITSS approach, established for foundation penetration in single soil layer, was extended to deal with spudcan penetration in multilayered clays by (Zheng et al., 2014). The extended LDFE/RITSS with the capability of simulating multilayer soils was employed in this thesis.

This chapter presents a review of the most commonly used FE approaches in solving large deformation problems of soils. Several FE formulations such as Lagrangian and Eulerian for large strain analysis are discussed briefly. The LDFE/RITSS approach and its key characteristics are discussed in detail, together with the extension of LDFE/RITSS method for multilayered soils.

### 3.2. FINITE ELEMENT TECHNIQUES FOR LARGE DEFORMATION PROBLEMS

#### 3.2.1. Lagrangian method and Eulerian method

For large deformation or large strain analysis, it is well known that the two main approaches are the Eulerian and Lagrangian formulations (Zhou, 2008). In the Lagrangian method, the finite element mesh is embedded in, and moves with the material constituting the continuum. The deformation path is approximated with increments in time. After each increment the reference configuration is updated with the solution, and then the elements are formulated again in the current state using updated nodal positions (McMeeking and Rice, 1975) in the mesh. Traditionally, large deformation problems in solid mechanics have been solved numerically by the finite element method using the Lagrangian method, as the governing equations in this...
method are relatively simple and the material properties, boundary conditions, and stress and strain states can be accurately defined. On the other hand the Eulerian method was initially introduced for FE applications to solve fluid mechanics problems, as the mesh is spatially fixed and the material flows through the mesh during simulation (Gadala et al., 1983, Gadala et al., 1984).

The pure Lagrangian approach, relative to the pure Eulerian approach, has the advantage of satisfying less complex governing equations. This may be attributed to the absence of the convection terms in the Lagrangian formulations, and also the simple updating techniques for path and history dependent materials. However, a significant limitation of this approach is encountered when the deformation of the material becomes extra large involving material rotation. The lack of control over the mesh movement results in distorted (sometimes entangled) meshes with large changes in element shapes and dimensions. This adversely affects the accuracy of the numerical solutions. Moreover, any simulations involving certain contact boundaries, especially those with sharp edges or corners, may not be represented precisely in this formulations. This is due to the fact that the boundary condition has to be specified on a material point which might move itself. Situations of this kind are frequently encountered in the numerical simulation of metal forming processes, e.g. extrusion, drawing etc., where the punch or die faces may have acute edges or abrupt surface discontinuities (Kiousis et al., 1988, Abu-Farsakh et al., 1998, Fischer et al., 2007).

3.2.2. Arbitrary Lagrangian–Eulerian (ALE) method

To circumvent the inaccuracies caused by the excessive mesh distortion in the large deformation analysis using the Lagrangian approach, a more flexible approach, termed Arbitrary Lagrangian–Eulerian (ALE) method, has been developed (Hughes et al., 1981, 3-3).
Chapter 3: Large deformation Finite Element methods and RITSS

Kawahara and Ramaswamy, 1988, Takashi and Hughes, 1992). It was initially applied to fluid domains and linear-path independent solids, as stress states only rely on the instantaneous displacement or velocity fields (Haber and Abel, 1983).

The ALE method combines the advantages of both Lagrangian and Eulerain approaches, as such the finite element mesh need not adhere to the material but may be in general motion relative to the material. The ALE method reserves the potential to represent a Lagrangian or Eulerian method as limiting cases at points, where such methods are desired. Thus, it is evident that an ALE method is ideally suited (sometimes even necessary) for solving complex problems in solid mechanics. Specific applications of this approach to geomechanics problems have been conducted to simulate cone penetration test in sand and clay (Susila and Hryciw, 2003, Walker and Yu, 2006).

This approach has been constantly developed to handle mesh distortion and entanglement (Nazem et al., 2006, Sheng, 2007). However the capability of the remeshing technique in the ALE method is very limited for modelling large penetration problems (Zhou, 2008).

3.3. RITSS AND FURTHER DEVELOPMENTS

3.3.1. RITSS method

In late 90’s, a more practical method for large deformation problems was developed using the H-adaptive mesh in RITSS (Hu and Randolph, 1998) method. The RITSS method essentially falls within the ALE category, and has been adopted in this study. As the name implies, the RITSS (remeshing and interpolation technique with small
strain model) method is a series of small strain analysis steps with intermediate remeshing steps to accommodate large soil deformations.

The ALE method, by decoupling nodal and material coordinates, was developed in order to minimize the limitations associated with pure Lagrangian or Eulerian method. Compared to ALE method, RITSS is simpler, more flexible and versatile, in particular for large penetration problems with complex constitutive models. RITSS method is based on the ALE concept but without the need to modify the basic finite element equations. Also, since the remeshing and interpolation procedures are essentially independent of the incremental analysis, it can be adopted with any existing finite element code. This has been evidenced by recent work by (Wang et al., 2015), showing that RITSS can be implemented to both ABAQUS (Hibbitt and Sorensen, 2000) and AFENA (Carter and Balaam, 2000).

The procedure in LDFE/RITSS method is shown in Figure 1. In this method an infinitesimal strain model is combined with fully automatic mesh generation and plane linear stress interpolation techniques. Remeshing and interpolation of historical variables are carried out after a specified number of small strain increments, with the total strain within one remeshing step kept in a small strain range. 50 incremental steps between each remeshing is recommended (Liu et al., 2005) and is used in this study. After a large deformation of soil occurs and the soil boundary becomes irregular, the regenerated mesh is made to fit the boundary of arbitrary shape very well, and excessive mesh distortion is successfully prevented. This method has been used to analyse large deformation problems of footings on single layered and double layered soil as well as the problem of spudcan footing penetration (Liu et al., 2005, Randolph et al., 2008, Yu et al., 2009).
In finite element analysis, the soil domain is subdivided into a mesh of discrete elements. The meshes used for the present footing problems were designed such that the elements were generally concentrated in the most highly stressed zones. The boundaries of the domain were sufficiently distant from the footing to minimise the domain boundary effect. Automatic optimal mesh generation is explored in section 3.3.3.

### 3.3.2. AFENA FE package

AFENA is a finite element program for the numerical analysis of boundary value problems in geotechnical engineering which was developed by Carter and Balaam at the University of Sydney (Carter and Balaam, 2000).

The compulsory files for AFENA calculation are list in the Error! Reference source not found. below:
Chapter 3: Large deformation Finite Element methods and RITSS

Table 1 Six compulsory files for AFENA calculation

<table>
<thead>
<tr>
<th>Finite Element Calculation</th>
<th>Afena.exe</th>
<th>AFENA executable file</th>
</tr>
</thead>
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<tr>
<td></td>
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<td>Dll files contains all the subroutines</td>
</tr>
<tr>
<td></td>
<td>Tetgen.dll</td>
<td>Dll files for Large Deformation analysis</td>
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<tr>
<td></td>
<td>Triangle.dll</td>
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</tr>
<tr>
<td>Pre-Processing</td>
<td>Tecpcove.exe</td>
<td>AFENA Analysis Studio</td>
</tr>
<tr>
<td>Post-Processing</td>
<td>Afenapos.exe</td>
<td>Post-Processing program</td>
</tr>
</tbody>
</table>

This software was used for geotechnical design based on small strain formulas. It has been used for research by academics in Australia and China. Since the users can obtain the source code, it became a good research tool with the capability to add or modify the code for specific research problems. The establishment of LDFE/RITSS (Hu and Randolph, 1998, 1998) is such an example. However, the pre- and post-processing of AFENA is not very simple and especially when large deformation problems are involved. Therefore, Dr. Liu and Dr. Yu in the Institute of Earthquake Engineering, Dalian University of Technology, China programmed and modified the tecpcove.exe and afenapos.exe files in order to process numerical results involving large deformation of soils with multi-layers (Liu et al., 2005).

This software, with extended capability of LDFE/RITSS analysis, has been used and proved to work well with single layered and double layered soil including sand over soft clay (Yu et al., 2009) and stiff clay over soft clay (Liu et al., 2005).
3.3.3. Mesh generation strategies (H-Adaptive Finite Element Method)

Although the RITSS technique has been proved to work well for many complex geotechnical problems, it was limited to one or two soil layered profiles. When it comes to the problems with spudcan penetration into multi-layered soil profiles, the quality insurance of every updated mesh after each remeshing is always an issue. However foundations with complex shapes penetrating into stratified soil seabed is quite common in reality. Therefore a modification was made by (Zhou et al., 2014) featuring a self-adjustable mesh updating technique which dramatically increased the stability of the updated mesh and enhanced the capacity of RITSS on analysing complex foundation deep penetration problems in three or more layered soil profiles.

The mesh used in this study comprises six-noded triangular elements with three internal Gauss points. A typical mesh generation strategy applied herein falls into the nodal connection class and is based on the two techniques listed below (Hossain, 2014):

(a) appropriate element connections was ensured by following Delaunay triangulation, using normal offsetting of the nodes from the nodes generated on the domain boundary

(b) Laplacian technique was used to adjust the obtained triangulated (majority right angular) element shapes, smoothing the mesh and producing optimum triangles (quasi-equilateral)

A typical example is shown in Figure 2. After the smoothing procedure (Figure 2b), the elements become more equal sided, which is a better shape for better numerical results.
However the region near the spudcan shoulder is a high strain zone where a denser mesh is needed to ensure the accuracy of the FE results.

(a) After offsetting and Delaunay triangulation  
(b) After smoothing procedure

Figure 2 Spudcan penetrating into soil with uniform mesh (Hu & Randolph, 1998b)

To optimise the mesh to achieve accurate FE results without increasing the total number of elements in the mesh, the mesh needs to be dense in the high strain zone and coarse in the low strain zone. The mesh density governing function used in this study, $f_d$, can be expressed as below:

$$f_d = Ae^{Ed}$$  

Equation 1

where $A$ and $E$ are constants and $d$ is the distance from the reference point on the structure, which is commonly assigned to the outer-edge of the foundation base. The effect of applying the mesh density governing function can be seen in Figure 3.
In order to control the density of the mesh automatically in each remeshing step, the h-adaptivity technique was introduced to the RITSS method by Hu and Randolph (1998). The h-adaptivity reduces the element size without changing the order of the polynomial expansions, and can be easily implemented in an existing finite element codes (Hu and Randolph, 1998). In their study a strain-SPR (Super-convergent Patch Recovery) criterion was also proposed to measure the strain error in an element expressed as follows:

\[ e_i^* = \left[ \frac{\int_{\Omega_i} (\varepsilon^* - \varepsilon^h)^T (\varepsilon^* - \varepsilon^h) d\Omega_i}{\Omega_i} \right]^{\frac{1}{2}} \]  

\text{Equation 2}

in which \( e_i^* \) is the strain error in element \( i \), \( \Omega_i \) is the area of element \( i \), \( \varepsilon^h \) are the strains from the FE solution, and \( \varepsilon^* \) is evaluated by SPR in a similar manner to the SPR of \( \sigma^* \) by (Wang et al., 2015). This measure represents the strain error in an element and the non-dimensional characteristic of this error estimator makes it convenient to use.

In this research a two dimensional axisymmetric mesh was adopted associated with six-noded triangular elements and three internal Gauss points. Following Hu & Randolph’s study (Hu and Randolph, 1998), the superconvergent value of local field strain recovered by SPR in the patch, \( \varepsilon^*_p \), can be evaluated as below:
\[ \varepsilon_p = Pa \]  
Equation 3

where the subscript \( p \) refers to the local patch, and \( P \) contains appropriate polynomial terms and \( a \) is a set of unknown parameters. In the present study, the quadratic terms for \( P \) and the six parameters in \( a \) are:

\[ P = [1 \ x \ y \ x^2 \ xy \ y^2] \]

\[ a = [a_1 \ a_2 \ a_3 \ a_4 \ a_5 \ a_6]^T \]

The unknown parameters \( a \) is determined by ensuring a least square fit of this to the set of strains from the FE analysis, \( \varepsilon^h \), and the recovered strains, \( \varepsilon^* \), using normalised coordinates \((x'_i, y'_i)\) of a group of sampling points. This requires minimisation of

\[ F(a) = \sum_{i=1}^{n} (\varepsilon^h(x'_i, y'_i) - \varepsilon^*_p(x'_i, y'_i)a)^2 \]

\[ = \sum_{i=1}^{n} (\varepsilon^h(x'_i, y'_i) - P(x'_i, y'_i)a)^2 \]  
Equation 4

\( n \) is the total number of sampling points in the patch. Obviously the minimisation condition of \( F(a) \) implies that \( a \) satisfies

\[ \sum_{i=1}^{n} P^T(x'_i, y'_i)P(x'_i, y'_i)a = \sum_{i=1}^{n} P^T(x'_i, y'_i)e^h(x'_i, y'_i) \]  
Equation 5

Therefore \( a \) can be solved by the following matrix form:

\[ a = A^{-1} b \]  
Equation 6

where

\[ A = \sum_{i=1}^{n} P^T(x'_i, y'_i)P(x'_i, y'_i) \]  
Equation 7

and

\[ b = \sum_{i=1}^{n} P^T(x'_i, y'_i)e^h(x'_i, y'_i) \]  
Equation 8

In summary the mesh refinement using \( h \)-adaptivity FE method is illustrated as below:

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3.3.4. Interpolation schemes

Large deformation analyses were undertaken using the RITSS (Remeshing and Interpolation Technique with Small Strain) approach developed by Hu & Randolph (Hu and Randolph, 1998) incorporating small strain analysis into large deformation analysis by frequent remeshing. After each small strain analysis cycle a new mesh using the updated/deformed boundary is generated and the field values, such as all stresses and material properties, are mapped from the old mesh to the new mesh by interpolation process.
In the stress interpolation process, Unique Element Method (UEM) / Modified Unique Element Method (MUEN) technique were employed. These techniques have been well explained and applied in previous studies (Hibbitt et al., 1970, McMeeking and Rice, 1975, Hu and Randolph, 1998) and the main steps are summarised below:

- A reference mesh is formed by updating the coordinates of the nodes and Gauss points of the old mesh according to the displacement of the preceding incremental step.
- Locate the element in the reference mesh which contains the particular Gauss point of the new mesh.
- Once the element is recognised, stress values were interpolated at the new Gauss point using the three Gauss points in the reference element.

This stress interpolation will follow either one of the following two paths:

If the new Gauss point falls entirely into the old Gauss points formed triangle zone (for example, G₁ shown in Figure 5, which enclosed by T₁, T₂, and T₃), the program will linearly interpolate the stress values at G₁ from the values at the old Gauss triangle (T₁T₂T₃).

If the new Gauss point lies outside the triangle zone formed by old Gauss points, (for example, G₂ shown in Figure 5), the following action will depends on the location of this new Gauss point. If this new Gauss point is close enough (within certain tolerance range) to one of the old Gauss point (for example, T₁ shown in Figure 5), the value of T₁ will be allocated directly to G₂. Otherwise the closest Gauss point from the neighbouring element (T₄ shown in Figure 5) will form a new triangular zone with the closest side of the old Gauss points (T₁T₂ shown in Figure 5).
Linearly interpolated stress value will be given to $G_2$ from the stress value at $(T_1T_2T_4)$.

![Figure 5 Interpolation of stress state by UEM/MUEM technique (Hibbitt et al., 1970)](image)

At the final stage of stress interpolation, the failure criterion must be checked and obeyed. This study mainly focus on clay type soils with low permeability, therefore Mohr-Coulomb soil model with a Tresca yield criterion is adopted.

Mohr-Coulomb model is an elastic-perfectly plastic model which is often used to model soil behaviours, including the stability of dams, slopes, embankments and shallow foundations and so on, due to its simplicity and the inclusion of plasticity. Strain hardening or softening effect of the soil are not considered in this model. Yielding starts when the deviatoric stress exceeds yield stress, following the equation as below:

$$\sigma_1 - \sigma_3 = 2s_u$$

Equation 9

3-14
where $\sigma_1$ and $\sigma_3$ are the maximum and minimum principal stresses respectively. $s_u$ is the undrain shear strength of soil.

3.3.5. New remeshing technique applied on multi-layered soil domain to avoid mesh disorder

The overall scheme of the RITSS approach is to divide the displacements of the foundation element into a number of incremental steps. And the displacements of the element in each step must be small enough to avoid gross distortion of the soil element. However when the soil domain involves different soil layers, this task becomes more challenging. The main challenges are summarized as below:

- With multilayered soil domain, soil boundaries will break and the break points need to be predicted accurately during deep penetration.
- One soil boundary which isolates two different materials can break multiple times, after which different soil materials need to be captured and allocated correctly.
- The foundation that is continuously penetrating from the soil surface will result in the formation of cavities in the soil above the foundation and subsequent soil back-flow into that cavity with soil from different layers. The new free surface of the soil needs to be recorded and updated correctly.

An improved remeshing technique for multi-layered soils was proposed by Dr. M. Zhou from Hohai University in 2012 (Zhou et al., 2012), which successfully provided the key to address the challenges listed above.

The technique developed by Zhou (2012) can capture the ever-changing soil boundaries dynamically and automatically without user intervention. When one layer of soil
material is thin enough at certain location (based on the criterion prescribed), the layer will be broken at the corresponding location to form two domains with a single layer material. To conserve the material volume, during the foundation deep penetrating into multi-layered soils, the soil layer can break by squeezing mechanism from $i$ segments into $i+1$ segments. When two adjacent soil boundaries are close enough ($\leq d_{cr}$, which is breaking criterion), the thin soil layer will break into two domains. After which, the soil material in breaking zone will be replaced by the adjacent soil material. The total number of soil material blocks increase and the whole process will be managed by the program dynamically (shown in Figure 6).

![Diagram showing soil boundary break](image)

**Figure 6** Soil boundary break process dominated by breaking criteria $d_{cr}$ (McMeeking and Rice, 1975)

In order to ensure the accuracy of results, dynamic displacement increment DT (a function of minimum mesh size $h_{\text{min}}$) is adopted. At the same time, a zone of mesh refinement is prescribed in certain areas to ensure the accuracy of the results, such as the areas near the foundation shaft and the foundation base. Meanwhile mesh density for the area which gradually becomes far away from the structure during penetration should be reduced to obtain a better displacement increment DT and higher calculation speed. In
Zhou’s study, the application of this dynamic adaptive remeshing technique in RITSS can be illustrated by the flow chart in Figure 7.

![Flow Chart](image)

Figure 7  Improved RITSS for foundation penetrating into multi-layered soils, following

(McMeeking and Rice, 1975)

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3.4. APPLICATIONS OF RITSS IN RECENT STUDIES

Nowadays the LDFE/RITSS method with remeshing technique has been used extensively for continuous penetration analysis of different objects, from simple object geometries and soil layer profiles, such as strip and circular foundations penetration in single and double layer soils, to complex objects such as spudcan foundation, suction caisson, ball penetrometer and T-bar penetrometer penetrating into multi-layered soils (Gao et al., 2012, Zhou et al., 2012, Ma et al., 2014, Zhou et al., 2014).

A recent example is the study of stiffened caissons penetrating into a single layer of non-homogeneous clay by (Zhou et al., 2015) using LDFE/RITSS method. Figure 8 shows a comparison between the physical model test by PIV imaging analysis and the LDFE/RITSS approach. It’s clear to see good agreements between the centrifuge test data and the LDFE results in terms of key features of the flow mechanisms: surface heave, formation of gaps above and between the embedded stiffeners, pattern and amount of infill soil into the gap between stiffeners.

![Comparison of soil flow mechanisms between LDFE analysis and centrifuge test data](image.png)

Figure 8 Comparison of soil flow mechanisms between LDFE analysis and centrifuge test data reported by (Hossain et al., 2012, Zhou et al., 2015)
For the stiffened caisson penetrating into multi-layered clays, Figure 9 shows the soil flow patterns for a caisson penetration in three-layer and five-layer deposits (Zhou et al., 2014). It can be seen that a considerable amount of top layer soil for both cases were trapped by the bottom caisson stiffener and the material boundary breaking points were successfully captured by the analysis of caisson penetration.

![Figure 9 Soil failure mechanisms for stiffened caisson penetration in three-layer and five-layer deposits (Zhou, et al, 2015)](image)

3.5. REMARKS

This chapter has firstly reviewed Lagrangian, Eulerian and ALE formulations for LDFE analysis. Also the RITSS method which will be adopted in this study has been discussed in detail. Several key aspects and developments related to this technique have been introduced, including the interpolation process and the remeshing strategy.

Comparing with other existing methods of large deformation analysis, such as Lagrangian, Eulerian and ALE, RITSS is conceptually and mathematically simpler,
more practical and more stable as only conventional small strain analysis is performed between the remeshing and interpolation steps. The method can potentially be implemented into any existing standard FE programs, where small strain analysis is essential, through user-written interface codes. Associated with the new remeshing techniques, the frequent remeshing and the interpolation of field quantities after each remeshing provide the capability of accommodating large changes in the geometry and soil properties of the soil domain. (Wang et al., 2015)

3.6. REFERENCES


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Chapter 3: Large deformation Finite Element methods and RITSS


CHAPTER 4 SPUDCAN PENETRATION IN STIFF UNIFORM CLAYS WITH AN INTERBEDDED SOFT LAYER

4.1 INTRODUCTION

As a popular type of offshore foundation, spudcans, supporting each leg of jack-up rigs, can have diameters ranging from 10 m to 20 m. When installing a spudcan foundation in stiff-over-soft clays, a sudden rapid penetration may occur after reaching the peak capacity in the top stiff layer. This sudden penetration, termed ‘punch-through’ failure, can occur during jack-up rig installation and operation stages, which may lead to severe structural damage of the jack-up rigs when one of the three legs of the rig experiences this sudden penetration. Two examples in recent years are the failures of Noble David Tinsley in May 2009 with severe damage to the legs and the rig (Noble, 2009), and the incident of Sapphire Driller in October 2009 during a preload test.

A continuous penetration of spudcan into the layered soil has been simulated numerically using the Remeshing and Interpolating Technique with Small Strain model (RITSS) in 2005 (Liu et al., 2005). Since then, numerical analysis has provided improved understanding of the spudcan punch-through failure in layered soils. The research on punch-through failure includes cases of spudcan on sand over soft clay (Yu et al., 2009) and stiff clay over soft clay (Liu et al., 2005, Hossain and Randolph, 2010).
using LDIF/RITSS method with AFENA (Carter and Balaam, 2000). It is apparent that, in the literature, most of the studies on punch-through failure are focusing on spudcans on double layered soils.

The behaviour of a spudcan penetrated into sand/clay/sand was investigated by Kellezi and co-workers (Kellezi and Stromann, 2003, Kellezi et al., 2005, Kellezi et al., 2008), using large deformation FE methods provided by ABAQUS Standard (Hibbitt and Sorensen, 2000), ABAQUS Explicit (Hibbitt and Sorensen, 2000) and ELFEN (R.S.Ltd.) FEM programs. However, there are many combinations of multi-layered soil profiles, e.g. stiff clay/soft clay/stiff clays, found in offshore field (Brennan et al., 2006). This stratified seabed sediments with potential for punch-through failure is typically encountered along Australia coast between water depths of 5 to 20 m below mud line, where most jack-up rigs are founded on spudcans (Erbrich, 2005).

This Chapter focuses on the cases where a soft clay layer is sandwiched in two uniform stiff clay layers, aiming to provide design charts for offshore engineers to design spudcan foundations on a seabed with this type of soil profiles.

4.2 NUMERICAL ANALYSIS

4.2.1 Geometry and parameters

The numerical model in this chapter has been considered as a spudcan foundation of diameter D, penetrating into two and three layered deposits as illustrated in Figure 1. The two layered cases are studied as a reference to examine the three layer effect. The model set up of a spudcan on a three layered clay is shown in Figure 2, including the dimensions of the typical spudcan used in this study. The boundaries of the soil domain in the model are sufficiently far enough from the foundation to minimise the boundary

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effect. An axi-symmetrical model is set up for the analysis. The soil domain extends to 20 times the foundation diameter (i.e. 20D) in both breadth and depth.

![Spudcan Penetration](image)

(a) double layered clay  (b) stiff clay/soft clay/stiff clay

Figure 1 Soil profile

To compromise the computational time and the accuracy of the analysis for practical use, the tip of the spudcan is initially embedded in the soil as shown in Figure 3. The zero penetration depths, \( d = 0 \), is defined where the widest brim of the spudcan just reaches the original soil surface level as shown in Figure 2. Before reaching this position, the penetration depth is shown as negative. The typical mesh at the initial setup is shown in Figure 3. The mesh will be refined to optimal by applying h-adaptivity before the large penetration analysis (please see Chapter 3 for details).
In the LDFE/RITSS analyses, all clay layers are assumed undrained. The uniform stiff clay layers at the top and bottom are identical with the same shear strength of $S_{\text{strong}}$. The strength of the stiff layer varies as $S_{\text{strong}} = 20$ and $40$ kPa. For the soft clay layer in the middle, the undrained shear strength varies as $s_{\text{um}} = 10$, $15$ and $20$ kPa. The relative thickness of the top stiff clay layer varies as $H_{t}/D = 1$, $2$, $3$ and the middle soft layer thickness varies as $H_{m}/D = 0.25$, $0.5$, $0.75$, $1$, $1.5$. The spudcan shaft is set as smooth (i.e. $\alpha = 0$, $\alpha$ is the friction coefficient). Spudcans are considered full rough (i.e. $\alpha = 1.0$) with diameters of $D = 10$, $14$ and $20$ m. Clay effective unit weight $\gamma'_{c} = 7$ kN/m$^3$ is selected for the underwater condition. The selected parameters for this study are summarised in Table 1.
Table 1 Summary of cases in LDFE simulations

<table>
<thead>
<tr>
<th>Analysis</th>
<th>D (m)</th>
<th>$s_{ut}/\gamma'D$</th>
<th>$s_{um}/s_{ut}=s_{um}/s_{ub}$</th>
<th>$H_{um}/D$</th>
<th>$H_{u}/D$</th>
<th>Note</th>
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</table>

4.2.2 Analysis details

Finite element analyses are conducted using the AFENA finite element package (Carter and Balaam, 2000) developed at the University of Sydney. The large deformation analysis was undertaken using the RITSS (Remeshing and Interpolating Technique with Small Strain model) method (Hu and Randolph, 1998) which essentially belongs to the ALE category (Ghosh and Kikuchi, 1991). As the name implies, the RITSS method is incorporating small strain analysis into large deformation analysis by frequent remeshing. Remeshing and interpolation of historical variables are carried out after a specified number of load steps, every 50 steps in this research. After large deformation occurs and the soil boundary becomes irregular, the mesh is regenerated (i.e. remeshing) to fit the boundary of the deformed shape, and excessive mesh distortion is successfully 4-5
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prevented. This method has been proved working well with large deformation problems of spudcan foundations on a double layered soil (Liu et al., 2005, Yu et al., 2009) as well as the problems on three layered soil (Yu et al., 2011).

The spudcan was considered rigid. The spudcan penetration was simulated by specifying incremental displacements of the spudcan. The size of the incremental displacement is set as 0.02% of the foundation diameter (i.e. \( DT = 0.0002D \)). (Hu and Randolph, 1998) showed that good accuracy can be obtained with incremental displacement of 0.1-0.2% of the foundation semi-width (or radius \( R \)), and remeshing every 10-20 increments of the analysis. In order to ensure the continuation of calculation, after balanced the sufficiency of accuracy and efficiency of time, \( DT = 0.001 \) m is selected (i.e. \( DT/D = 0.01 \sim 0.005 \) for the spudcan diameters considered) and remeshing is implemented at every 50 increments in this research.

Due to the undrained condition for clay in total stress analysis, both soil friction and dilation angles are defined as 0° and a uniform stiffness ratio \( E/s_u = 500 \) is selected. The saturated unit weights of clays are all taken as \( \gamma_c = 17\text{kN/m}^3 \) and water unit weight, \( \gamma_w \) is 10kN/m\(^3\). Thus the effective soil unit weight, \( \gamma' \) is 7kN/m\(^3\) is used in the analyses for underwater conditions. These parameters are used for all the clay materials in this study.

4.3 EFFECT OF BOTTOM STIFF LAYER ON SOIL FAILURE MECHANISMS

Soil resistance to the penetrating spudcan and the soil failure mechanisms during the spudcan penetration are closely related. In this section, the soil flow mechanisms during spudcan penetration into two-layers of stiff-soft clays and three layers of stiff-soft-stiff
clays are compared. The impact of the bottom stiff layer on the soil failure mechanisms is examined.

Figure 4 depicts a series of typical soil flow mechanisms during a spudcan penetration in two and three layered clays with strength ratio of $s_{um}/s_{ut} = s_{um}/s_{ub} = 0.25$ (Group I-1-2, $H_m/D = 0.5$ in Table 1). The lower strength ratio has the potential to induce more severe pouch-through failure, and to show more apparent effect of the bottom stiff layer.
From Figure 4, it can be seen that, for both two-layer and three-layer soils, at the beginning of the spudcan penetration (d/D = 0.3, Figure 4a), the top stiff layer is
bending downwards to the underlain soft layer. The soft second layer shows a general bearing failure in the two-layer soil (left graph of Figure 4a). However, the second soft layer shows clear horizontal movements in the three-layer soil (right graph in Figure 4a) due to the presence of the bottom stiff layer.

With further penetration of the spudcan as \( d/D = 1.2 \) in Figure 4b, a reverted cone made of the top stiff clay is formed underneath the spudcan and forced into the underlain soft layer with the spudcan. Compared to the double-layer case, this reverted cone is much smaller in the three-layer case due to the squeezing mechanism in the middle soft clay, which also induce more horizontal movements of the inverted cone due to the presence of the bottom stiff clay. In the bottom stiff layer, it behaves like a rigid boundary and no soil movements can be observed.

Once the tip of the spudcan touches the bottom stiff layer \( (d/D = 1.5 \) right graph in Figure 4c), soil in this layer starts to move downwards slightly while most of the soil movements are still confined in the top two layers. For the two-layer case, the inverted cone stays underneath the spudcan unchanged (left graph in Figure 4c). However in the three layer case, the middle soft layer is squeezed out horizontally between the top and bottom stiff layers. Thus a clear shear surface between the top stiff soil and the middle soft soil is formed.

At the final stage of spudcan foundation penetration shown in Figure 4d, the spudcna is fully embedded in the soil with soil backflow onto the top of the spudcan, with an open cavity formed near the soil surface. The open cavity in the three-layer case is slightly shallower but similar to that in the two-layer case due to the squeezing effect. For the two-layer soil case, well defined stiff clay plugs above and below the spudcan foundation can be observed moving downwards with the foundation during penetration.
However the three-layer case, a much smaller soil plug with the top stiff soil formed underneath the spudcan. This is due to the “squeezing” mechanism in the second soft layer. The combined effects of squeezing in the soft layer and the ‘rigidity’ provided by the bottom layer make the top layer move around the spudcan more easily, which reduces the soil plug size underneath the spudcan and the cavity above the spudcan. Once the small soil plug with the top stiff soil meets the bottom stiff soil layer, the spudcan is fully embedded in a uniform clay layer, since the top and bottom clays are identical. The effect of middle soft soil layer diminishes at this stage.

Current offshore design guidelines (ISO, 2012) provide an assessment method based on the squeezing mechanism of a soft layer between a foundation and a stiff soil layer (see Figure 5 with the layer 2 being the soft layer) following Meyerhof & Chaplin (1953). However this approach was developed on the basis of model tests conducted directly on the surface of a soft cohesive layer. Therefore this approach does not account for the distortion of the top layer as it punches through to the soft layer. From Figure 4 it can be seen that the squeezing mechanism occurs between two stiff soil layers rather than between foundation and stiff soil layer. The deformation (or bending) and rotational flow of the top stiff layer combines with squeezing mechanism of the middle soft layer. Therefore, it is imperative to study the spudcan response via its continuous penetration in stiff-soft-stiff soil profiles.
4.4 CONTINUOUS PENETRATION BEHAVIOUR OF SPUDCAN FOUNDATIONS

4.4.1 Spudcan in single uniform clay and double uniform clay– Calibration

In order to validate the numerical results in this study, the bearing pressure of a spudcan penetrating into a uniform clay deposit is compared with the ultimate bearing capacity factor obtained by previous research. The soil profile is divided into three layers with $H_1 / D = H_2 / D = 1$, but with all layers having $s_u = 20$ kPa. The bearing capacity factors for the penetrating spudcan with both smooth and rough interfaces are graphed in Figure 6. The ultimate bearing capacity factor reached in this analysis is $N_c = 13.15$ for the rough spudcan, which is marginally smaller ($\sim 2\%$) than ultimate capacity factor of $N_c = 13.44$ by (Liu et al., 2005). This small difference may be attributed to the finer mesh near the spudcan shoulder used in this study than the by Liu et al. (2005). The equivalent penetration using a smooth spudcan yielded an 11% lower ultimate bearing capacity of $N_c = 11.70$. This is also comparable to the results observed in centrifuge test by Hossain 4-11
and his co-workers (Hossain et al., 2003) since the spudcan model in centrifuge test has always been polished and more close to a smooth interface. A fully smooth spudcan is analysed for all following cases in this study (except for validation cases).

As part of the model calibration, a spudcan penetrating into stiff-soft double layered uniform clays is also studied. The peak bearing capacity factors of the spudcan are compared with those from the previous research in Figure 7, including a flat circular footing case. The clay layer strength ratios, $s_u/s_{ub}$, varied from 0.1 to 1.0, where the top clay layer thickness is the same as the spudcan diameter ($H/D = 1.0$) and the spudcan diameter $D$ is 14 m. It can be seen that all spudcan bearing capacity factors are higher than those of the flat circular footing. This is because (a) the spudcan has a conical base.

![Figure 6 Validation of FE results of spudcan penetration in uniform clays](image)

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and the circular footing has a flat base; (b) the spudcan allows soil to flow back on top of it after the limiting cavity depth is reached and the circular footing has vertical shaft along the periphery of the footing, thus there is now allowance for soil back (Hossain and Randolph, 2010). (Wang and Carter, 2002) have reported 20% higher capacities for spudcan foundations than that of circular footings, which is also observed here.

The peak spudcan bearing capacity factors from current analysis are between the two groups of results by Liu, et al. (2005). Although all spudcan analysis are for D = 14m, the top clay layer strengths varied from 10 kPa to 100 kPa with 40 kPa in the current analysis. Under the same soil strength ratios, the peak spudcan bearing capacity factor increases with decreasing top layer strength $s_{ut}$. This is because a softer top clay layer (i.e. lower $s_{ut}$) generates a shallower cavity (Hossain et al., 2005), hence the spudcan capacity reaches its fully embedded state earlier and the bottom layer becomes less effective. Thus in uniform soil ($s_{ut}/s_{ub} = 1.0$), the peak spudcan capacity factor (or ultimate capacity factor) is not affected by the clay strength, which is also shown here.

![Figure 7 Spudcan peak bearing capacity factors for H/D=1](image)

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4.4.2 Limiting cavity depth in top stiff layer

From the soil flow mechanisms corresponding to continuous penetration of spudcan foundation in stiff-soft-stiff layered soils, an open stable cavity with vertical walls can be observed after the foundation is fully embedded as shown in Figure 8. The (average) limiting cavity depth which influences the bearing capacity, uplift capacity and moment resistance of the foundation can be described as a function of soil parameters and layer geometry ($kD/\gamma uH/D, s_u/\gamma' D$) based on the previous study by (Hossain et al., 2005).

For a single layer of uniform clay, the expression for limiting cavity depth was proposed by (Hossain et al., 2005) given below:

$$\frac{H}{D} = \left( \frac{s_uH}{\gamma' D} \right)^{0.55}$$  \hspace{1cm} (1)

where $H$ is the depth of the stable cavity formed during deep penetration and $s_uH$ is the undrained shear strength of the single soil layer. This formula was expanded to double layered clays by (Hossain and Randolph, 2010) with a uniform stiff clay layer over a soft normally consolidated clay layer. The limiting cavity depth of a spudcan foundation penetrating into the double layer clay was also proposed as follows:

$$\frac{H}{D} = 1.4 \left\{ \left( \frac{s_uH}{\gamma' D} \right) \frac{t}{D} \left( 1 + \frac{kD}{\gamma' u_s} \right) \right\}^{0.5}$$  \hspace{1cm} (2)

where $t$ is the top layer thickness and $k$ is the strength gradient of the bottom layer.

Based on all the cases studied as tabulated in Table 1, a design chart for cavity depth prediction in stiff-soft-stiff clays is established in Figure 10. The normalised cavity depth $H/D$ is plotted against a non-dimensionalised factor of $\left\{ \left( \frac{s_uH}{\gamma' D} \right)^{H_m/D} \left( \frac{s_uH}{s_{u_m}} \right)^{0.5} \right\}^{0.36}$, where $s_u/\gamma' D$ is the normalised shear strength of the top layer, $H_m/D$ is the normalised
middle soft layer thickness and $s_{ut}/s_{sum}$ is the strength ratio of stiff and soft layer. Although the normalised strength gradient $kD/su$ has been proved to affect the cavity depth as shown in Equation 2 by Hossain and Randolph (2010b), all the cases presented in this study are with layered uniform clays (i.e. $k = 0$). Therefore $kD/su$ is not included here. Besides the strength ratio of the top to middle layer clays equals to that of bottom to middle layer as $s_{ut}/s_{sum} = s_{ub}/s_{sum}$ (i.e. $s_{ut} = s_{ub}$).

![Figure 8 limiting cavity depth](image1.png)

![Figure 9 Cavity depths for double layered clay from centrifuge tests and LDFE (Hossain and Randolph, 2010)](image2.png)
Figure 10 Cavity depth H during spudcan penetration in stiff-soft-stiff clay

The soft clay layer sandwiched between two stiff layers has a great influence on the cavity formation in the top stiff layer. As discovered previously for spudcan penetration in single and double layered clays (Hossain et al., 2005), the cavity collapse is not due to the cavity wall stability, it is due to the downwards movement of the spudcan to push the soil underside the spudcan flowing around the spudcan towards its topside. Thus the relatively thick underlying soft middle layer will encourage underside soil to move downwards, rather than pushing the stronger soil to flow back towards its topside. This can delay the initiation of soil backflow. Hence the cavity depth of H/D increases with increasing middle layer thickness of H_m/D and strength ratio of s_m/s_sum (i.e. decreasing middle layer strength). Consequently the cavity depth for three-layer cases are lower than the design curves proposed for double layered soil and get closer to that for double layer soils as the middle layer thickness of H_m/D increases. The limiting cavity depth for spudcan with different diameters eventually remains constant that matches well with the design formula proposed previously for double layer clays (Eq. 2).

The best-fitted trend line from the LDFE results of spudcan on stiff-soft-stiff clays can be expressed as below (i.e. line T-T indicated in Figure 8):

\[
\left(\frac{s_{uD}}{\gamma D}\right)^{0.36} \left(\frac{H_m}{D}\right)^{0.5} \left(\frac{s_m}{s_{sum}}\right)^{0.36} = 0.4
\]
\[
\frac{H}{D} = 1.15\left\{ \frac{S_u H}{\gamma H D} \left( \frac{H_m}{D} \right) \left( \frac{S_{ut}}{S_{um}} \right)^{0.5} \right\}^{0.36}
\]

Once the middle layer is thick enough, the bottom layer effect diminishes, and the normalised cavity depth becomes horizontal lines (i.e. DD1, DD2 and DD3 in Figure 8). These horizontal lines are consistent with the trend proposed by Hossain and Randolph (2010a) as shown in Figure 8. This design graph unifies the cavity depth predictions for two layer and three layer clays. It can be used for cavity depth assessment of spudcans on stiff-soft and stiff-soft-stiff clays.

### 4.4.3 Bearing response

The load-penetration responses during spudcan deep penetration are presented in terms of bearing capacity factor \(N_c\) which is calculated using:

\[
N_c = \frac{F_{net}}{A_s u} = \frac{F_{total} - F_{buoyancy}}{(D/2)^2 \pi s_u}
\]

where \(F_{total}\) is the total reaction on the foundation which can be obtained from the analysis and \(A\) is the largest projected area of the spudcan foundation \(A = (D/2)^2 \pi\). \(F_{buoyancy}\) is the buoyancy when spudcan is embedded in the soil. The strength of top clay layer is used as \(s_u\) in this equation. Spudcan weight is not included here, thus it is weightless.

The bearing response of spudcan foundations on multi-layered clay is affected by a number of factors. However the following four factors are seen playing major roles in spudcan penetration responses, and thus discussed in more detail. The four major influencing factors are:

1. middle soft layer thickness ratio - \(H_m/D\)
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2. top stiff layer thickness ratio - $H_t/D$

3. soil layer strength ratio - $S_{um}/S_{ut} = S_{um}/S_{ub}$ (since $S_{ut} = S_{ub}$)

4. spudcan diameter - $D$

The effects of these four parameters are evaluated based on the LDFE/RITSS analysis and all cases are summarised in Table 1. Please note that in practice the diameters of spudcans normally ranging from 10 m – 20 m and here three different diameters (10m, 14m, 20m) are selected to study the foundation size effect. $D = 14$ m is selected since it is the commonly used size in practice (ISO, 2012).

4.4.4 Effect of middle soft layer thickness ratio - $H_m/D$

In order to demonstrate the effect of the relative thickness of the middle soft layer, bearing resistance profiles are plotted in Figure 11 for a range of $H_m/D = 0, 0.25, 0.5, 0.75, 1, 1.5, \infty$ with the strength ratio kept constant at $S_{um}/S_{ut} = S_{um}/S_{ub} = 0.25$. When $H_m/D = 0$, it means that the soil profile is a single layer soil since $S_{ut} = S_{ub} = 40$ kPa, and when $H_m/D = \infty$ it means that the soil profile is a double layered soil with strength ratio $S_{ub}/S_{ut} = 0.25$. 

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Figure 11 Effect of the relative thickness of middle soft layer on spudcan bearing response (Group I-1-2 in Table 1: $H_m/D=1$, $s_{um}/s_{ult}=s_{um}/s_{uhb}=0.25$ $D=14m$)

It can be seen from Figure 11 that, when the middle soft layer is thin as $H_m/D = 0.25$, there is no peak bearing capacity observed. Thus there is no punch-through failure risk. Although the spudcan bearing capacity reduces when it passes the middle soft layer, it quickly catches up the bottom stiff layer and the bearing resistance merges with that of single clay layer.

When the middle soft layer is thick as $H_m/D > 0.25$, there is a peak resistance followed by a resistance reduction when the spudcan penetrates through the middle soft layer. However, when the spudcan is moving closer to the bottom strong layer, the bearing resistance starts to increase again. After the spudcan had penetrated through the whole soft layer, the bearing capacities all went back and finally, the bearing capacities for all the cases considered can reach the same value as single layer case with $N_c = 13.2$ which

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is 2.5% below the resistance from Liu’s study (2005). The current resistance of $N_c = 13.2$ matches well with previous centrifuge tests by (Hossain and Randolph, 2009).

With increasing middle soft layer thickness ratio, $H_m/D$, the peak bearing capacity decreases, due to larger effect of the soft middle layer. Because of the thicker soft layer, it takes more penetration depth to mobilise the full resistance from the bottom stiff layer. Thus, the punch-through depth increases.

The bearing responses in three layers transit between the single layer response (i.e. $H/D = \infty$) and the double layer response (i.e. $H_m/D = \infty$). The bearing responses in three layers are present when both $H/D$ and $H_m/D$ are limited. Hence in three layered soils with stiff-soft-stiff profiles, the bearing response of the spudcan experiences the double layered effect (stiff-soft) initially. Once the spudcan penetrates into the bottom stiff layer, the effect of the middle soft layer diminishes with further penetration. Eventually, the middle soft layer is squeezed out and the top and bottom stiff clays join together (see Figure 4d), and the spudcan bearing response is only dependent on the stiff soil strength.

From Figure 11, it is apparent that a peak foundation capacity can be identified when the middle soft layer is thick enough as $H_m/D \geq 0.5$. The bottom stiff layer affects the spudcan peak capacity when $H_m/D = 0.5$. However, the influence of the bottom stiff layer to the peak capacity fades quickly when $H_m/D \geq 0.75$. With further penetration of the spudcan, the influence of the bottom stiff layer becomes obvious when the spudcan response deviate from the double layer curve. All bearing responses converge to a single layer response after the foundation passes its original layer interface by 0.8D.
For the cases with $H_m/D \geq 0.75$, the peak capacity of the spudcan in the three layered soil can be predicted using the double layered response (SNAME, 2008, Hossain and Randolph, 2009). The punch-through depth can be estimated as the peak capacity is regained due to the bottom stiff layer effect after the capacity reduction occurred. Therefore it is important to establish the transition curves between the double layer response and the single layer response for spudcan penetrating in three layered soils.

4.4.5 Effect of the top layer thickness relative to the spudcan diameter ($H_t/D$)

In order to study the effect of relative top layer thickness, the bearing responses for Group III-1 in Table 1 are plotted in Figure 12. Most of soil parameters used here are identical to the ones used in Figure 11. The only variable of the relative top layer thickness ($H_t/D$) is changed from 1 in Figure 11 to 3 in Figure 12.
Figure 12 Effect of the relative thickness of top stiff layer on bearing response (Group III-1 in Table 1: \(H_t/D = 3\), \(s_{top}/s_{sat}=s_{top}/s_{sub}=0.25\) \(D=14m\))

It can be seen from Figure 12 for \(H_t/D = 3\) that spudcan penetration responses show similar patterns to those for \(H_t/D = 1\) cases in Figure 11. However due to the increase top layer thickness to \(H_t/D = 3\), the bearing capacities for all the cases of the spudcan in the top layer reach its stabilized state and then followed by a reduction due to the underlying soft material. With thicker top layer, the spudcan foundation has enough space to reach the full capacity with full rotational soil flow around the foundation before the next soft layer affects the bearing capacity greatly. It is more evidence to see all the cases with three layered soils range nicely between the single layer case and the double layer case as the Group I-1-2 results shown in Figure 11 while the gap between lower bound and upper bound is much narrower in the top layer compared to \(H_t/D=1\) cases in Figure 11. Since all the cases in this group reached full capacity, even a thin 4-22
interbedded soft material \( (H_{m}/D = 0.25) \) will result in a bearing capacity reduction which is termed as punch-through failure. On the other side when \( H/D = 1 \), no punch-through failure potential can be observed in Figure 11 with \( H_{m}/D = 0.25 \).

The penetration response of spudcan foundations is directly corresponding to the soil flow mechanism around the foundation. For instance of \( H_{m}/D = 0.75 \), the soil flow at 4 different positions (Peak, Trough, Recovery, Final state) marked by "■" in Figure 12.

It is clear from the soil flow plots that at point 1, foundation starts to feel the middle soft layer; point 2 presents a full flow mechanism with large contribution from the middle soft layer; point 3 shows the middle soft layer is gradually squeezed away from the spudcan; and finally, the full flow mechanism is dominated by the stiff soil from the top and bottom layers at point 4. It is much clearer that at point 1 a full rotational flow has been generated and the peak value appears when the soft material shows a sign of movement and start to affect the bearing capacity. At point 2, due to the squeezing mechanism in the middle soft layer, most part of the shear plane passed through the soft clay layer leading to a reduced bearing capacity. With a deeper penetration at point 3, the soft clay is squeezed out and the top and bottom stiff layers join together, where the shear plane passes all three layers the bearing response start to recover. At the final state of point 4, all the soft material is squeezed aside and a new shear plane is generated which passes through the joined top the bottom stiff clays. Hence, the bearing capacity reaches a newly found stabilized value, which is same as the bearing capacity observed for the single layer case due to the indentical clays in the top and bottom layers.
### 4.4.6 Effect of strength ratio \((s_{um}/s_{ut} = s_{um}/s_{ub})\)

In order to demonstrate the effect of the strength ratio \((s_{um}/s_{ut} = s_{um}/s_{ub})\), bearing resistance profiles are plotted in Figure 13 for a range of \(s_{um}/s_{ut} = s_{um}/s_{ub} = 0.25, 0.375, 0.5, 0.75, 1\). \(H_m/D = 0.25, 0.5, 1\) with the same size of spudcan foundation with \(D = 14m\) (Group I-1-2, Group I-2-2, Group I-3-2 in Table 1). The ratio \(s_{um}/s_{ut} = s_{um}/s_{ub} = 1\) represents a single layer of uniform soil with strength \(s_u = 40\) kPa. Double-layer cases for different soil strength ratios are also presented for reference with the three layered soil profiles plotted for discussion.

![Figure 13](image)

(a) \(H_m/D=0.25\)  
(b) \(H_m/D = 0.5\)  
(c) \(H_m/D = 1\)

**Figure 13** Effect of strength ratio \((s_{um}/s_{ut} = s_{um}/s_{ub})\) on bearing response at \(H/D=1, D=14m\) (Groups I-1-2, I-2-2 and I-3-2 in Table 1)

For all three layered cases, it can be seen from Figure 13 that with increasing soil strength ratio (from 0.25 to 0.75), the punch-through potential reduces. Punch-through failures are observed for the case in Figure 13 with strength ratio \(s_{um}/s_{ut} = s_{um}/s_{ub} = 0.25, 0.375, 0.5\). Once \(s_{um}/s_{ut} = s_{um}/s_{ub}\) reaches 0.75 for all three \(H_m/D\) cases, there are no failures.
obvious peak bearing capacity observed. The overall bearing capacity is increasing with increasing soil strength ratio. This leads to different peak and transit capacities in the middle soft layer at slightly different penetration depth at the same Hm/D case. The penetration depth at peak capacity is decreasing with increasing soil layer strength ratio. However, since the penetration depths for both peak and transit capacities are changing in the same fashion, the strength ratio has minimum effect on the punch-through depth.

It is also noticeable especially from Figure 13b that the higher strength ratio sum/sub = sum/sum = 0.5, relative to sum/sub = sum/sum = 0.25 and 0.375 delays the timing when the foundation can sense the stiff bottom layer and start to increase its bearing capacity. This can be explained based on the soil flow mechanisms in Figure 14 for the case of H/D = 1, Hm/D = 0.75 and D =14 m. In Figure 13b, the bearing response curve in the top two layers for three layered clay is higher than the corresponding double layer case with strength ratio sum/sub = sum/sum = 0.25. However, at the strength ratio of sum/sub = sum/sum = 0.5, the bearing response in the top two layers for the three layered case is much closer to its corresponding double layer case. In Figure 14, for sum/sub = sum/sum = 0.25, the soft layer starts to deform almost immediately after the widest brim of spudcan touches the soil surface while that with higher strength ratio hasn’t shown the same response (Figure 14a). This means the softer middle clay layer allows the top layer to bend downwards more easily.
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Figure 14 Soil failure mechanism for spudcan penetration in stiff-soft-stiff clay at $H_m/D=0.75$

$s_{um}/s_{ut}=0.25$  $s_{um}/s_{ut}=0.5$  $s_{um}/s_{ut}=0.25$  $s_{um}/s_{ut}=0.5$

(a) $d/D=0.3$  (b) $d/D=1.2$

(c) $d/D=1.7$  (d) $d/D=2.4$

With further penetration of the spudcan, a larger stiff clay plug under the spudcan is formed with the lower strength ratio of 0.25 and is trapped down continuously leading...
to a significant squeezing mechanism in the middle layer (Figure 14b). Thus, a relatively thinner middle layer is left between the two stiff layers. The large stiff clay plug for the case with low strength ratio enables the foundation to sense the bottom layer earlier. The bottom stiff clay doesn’t show any movement until the middle soft clay is squeezed out and the spudcan touches this bottom layer (Figure 14c and d). After the two stiff clay layers join together, the spudcan bearing capacity converges towards the single layer response (Figure 11). This explains why the strength ratio does affect the penetration depth of spudcan peak capacity and the transit capacity to reach the bottom layer response in a same rate. Thus it doesn’t show much effect on the punch-through depth in this study.

When the penetration reaches to the final steady stage (Figure 14d), all the soft soil underneath the spudcan has been squeezed sideways and moves to the top of the spudcan. The deformations of the bottom layers at both strength ratio cases are almost identical. Since the top clay and bottom clay are identical, the subsequent penetration of the spudcan is expected to behave the same as in a single stiff clay layer.

![Design chart for punch-through depth for stiff-soft-stiff soil profiles](image)

\[ \frac{H_{pt}}{D} = \frac{H_m}{D} - 0.8 \times \frac{H_s}{D} + 0.3 \times \frac{\gamma_D}{s_w} - 1.9 \]

\[ H_m = \frac{H_s}{D} + 0.8 \times \frac{H_s}{D} + 0.3 \times \frac{\gamma_D}{s_w} \]

**Figure 15** Design chart for punch-through depth for stiff-soft-stiff soil profiles
Since the strength ratio shows a minimal effect on the punch-through depth, a design chart for punch-through depth \((H_{pt}/D)\) is proposed based on the other three major factors: \(H_m/D\), \(H_t/D\) and \(\gamma'D/s_u\). Results on the punch-through depth from all the cases studied are presented in Figure 15. In this figure, the normalised punch-through depth of \(H_{pt}/D\) is plotted against normalised factof of \(H_m/D+0.8H_t/D+0.3\gamma'D/s_u\), where \(s_u\) is the strength of the top layer in this study. The definition of the punch-through depth is the length from the penetration depth of the peack to the penetration depth to recover this capacity during the capacity transition stage as illustrated in Figure 15. Thus for a spudcan penetrating into stiff-soft-stiff clays, the punch-through depth can be expressed as:

\[
\frac{H_{pt}}{D} = \frac{H_m}{D} + 0.8\times \frac{H_t}{D} + 0.3\times \frac{\gamma'D}{s_u} - 1.9 \rightarrow for \ \left(\frac{H_m}{D} + 0.8\times \frac{H_t}{D} + 0.3\times \frac{\gamma'D}{s_u}\right) > 1.9
\]

\[
\frac{H_{pt}}{D} = 0 \rightarrow for \ \left(\frac{H_m}{D} + 0.8\times \frac{H_t}{D} + 0.3\times \frac{\gamma'D}{s_u}\right) \leq 1.9
\]

This equation is summarised based on all the cases available in the current study \((H_t/D = 1~3, \ H_m/D = 0.25~1.5, \ s_u/s_{sat}=s_{sum}/s_{sub}=0.25~0.75)\). In principle, this relationship is only suitable where the top stiff clay layer and the middle soft clay layer are thick enough for punch-through failure. When the top stiff layer or the middle soft layer is too thin, no peak value will be observed and the \(H_{pt}/D=0\) as marked in the plot.

### 4.4.7 Effect of the foundation size (D)

Although the clay layer thickness and other variables are normalised by the spudcan diameter \(D\), it is not very clear if the absolute foundation has any effect on the results shown above. The effect of spudcan diameter is shown in Figure 16 with \(H_t/D = 1.0\), strength ratio \(s_{sum}/s_{sat} = s_{sum}/s_{sub} = 0.25\) and \(H_m/D = 0.25, 0.5\). It can be seen from Figure 16...
that at strength ratio of $s_{um}/s_{ut} = s_{um}/s_{ub} = 0.25$ and $H_m/D = 0.5$, the peak spudcan capacity increases with increasing spudcan diameter and the larger diameter spudcan displays higher punch-through failure potential especially for both cases of $H_m/D = 0.25$ and $0.5$. At $H_m/D = 0.25$, the peak spudcan capacity can only be observed for $D = 20$ m. This means there is no punch-though failure potential for spudcans with smaller diameters ($D = 10$ and $14$ m). Therefore, by increasing the spudcan diameter, the peak spudcan resistance is increased. However, at the same time, the punch-through failure potential is increased as well. As the penetration reach the third layer of soil, the residual bearing capacities for these three different diameters ($10$ m, $14$ m, $20$ m) are the same with $N_c = 13.2$. The increased potential of punch-through failure with increased spudcan diameter can be observed more easily for the case of $H_m/D = 0.5$ in Figure 16.
Figure 16 Effect of the foundation size \( D \) on bearing response: \( H_m/D=0.25,0.5, H/|D|=1 \),
\( s_{us}/s_{us}=s_{us}/s_{sub}=0.25 \) \( D=10m,14m,20m \)

Figure 16 also depicts the effect of spudcan diameter on soil deformation during large penetration with \( d/D = 0.8 \) and 1. The figures are scaled to the same relative size for comparison. It can be seen that the stiff soil trapped around the spudcan are almost identical for both spudcans of \( D = 10 \) and 20 m. However, the depth and timing of limit cavity formed above the spudcan are different due to the different soil strength ratio of \( s_{us}/\gamma’H \).

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The relative limit cavity depths ($H_c/D$) are higher for smaller spudcans (Equation (3)). For spudcan of $D = 10$ m with $S_{soft}/S_{strong} = 0.25$, the cavity doesn’t start to be backfilled at penetration depth $d/D = 0.8$. However, for the same case with larger diameter ($D = 20$ m), the soil backflow onto the top side of the spudcan can be observed in Figure 16. Correspondingly, the bearing response in Figure 16 ($H_m/D = 0.25$) becomes constant before an obvious reduction with the maximum resistance occurring just before the spudcan reaches the middle soft layer. The case where punch-through occurs ($D=10$ m $H_m/D=0.25$ Figure 16) shows the bearing capacity increasing monotonically up to a plateau. It became clearer for the soil flow mechanism at the $D = 10$ m case, the tip of spudcan foundation has touched the bottom layer and the stiff soil has contributed to the bearing resistance when the soil above the foundation just starts to move towards the cavity avoiding any reduction in bearing capacity.

4.5 REMARKS

This chapter has reported results of LDFE analyses investigating the potential for punch-through failures of spudcan foundations during deep penetration through three-layered uniform clays with a soft layer sandwiched in the middle of two stiff clay layers. The LDFE analyses are conducted allowing the spudcan foundation to penetrate continuously from the sea-bed surface. The resulting soil flow mechanisms and profiles of penetration resistance are presented to discuss the effects of a wide range of normalised soil properties and layer geometry, especially focusing on the likelihood of spudcan punch-through and its severity.

The results show that the penetration resistance of spudcan foundations in three layered uniform clays depends on a number of dimensionless factors.
(a) the thickness of middle layer relative to the spudcan diameter, $H_m/D$

(b) the thickness of top layer relative to the spudcan diameter, $H_t/D$

(c) the strength ratio between middle and top layers (the strength ratio between middle and bottom layers) $s_{un}/s_{ut}$

(d) size of foundation, $D$

The parametric study covered a practical range of these parameters.

The effects of clay layer thickness are:

- At constant $H_t/D$, the peak bearing capacity of spudcan decreased with increasing middle layer thickness of $H_m/D$;
- With a thinner top layer ($H_t/D = 1$), the potential of punch-through failure reduced with decreasing middle soft layer of $H_m/D$. The potential of punch-through failure reduced to none at very thin middle layer thickness of $H_m/D = 0.25$;
- With an increased top layer thickness to $H_t/D = 3$, the potential of punch-through failure was increased, as the thin middle soft layer (e.g. $H_m/D = 0.25$) could trigger the punch-through failure.

The effects of soil strength ratio are:

- The overall spudcan bearing capacity increased with increasing soil strength ratio of $s_{un}/s_{ut}=s_{un}/s_{ub}$;
- At the soil strength ratio of $s_{un}/s_{ut}=s_{un}/s_{ub} = 0.75$, there was no obvious peak capacity observed. Hence there was no punch-through failure potential;
- With reduced soil strength ratio of $s_{un}/s_{ut}=s_{un}/s_{ub}=0.25$, 0.375 0.5, punch-through failure was observed for these cases. However, the punch-through failure depth was constant for all these cases. Therefore, a formula to prediction punch-through failure depth was established (i.e Eq. 3) with all other factors except the soil strength ratio.

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Chapter 4: Spudcan Penetration in Stiff Uniform Clays With an Interbedded Soft Layer

The spudcan diameter was found to have an effect on the spudcan behaviour. When all the normalised factors (i.e. $H_0/D$, $H_m/D$, $s_{ut}/s_{um}$) were kept constant, by increasing the spudcan diameter, the peak spudcan resistance was increased (i.e. positive effect). At the same time, the potential of punch-through failure was also increased (i.e. negative effect). Thus caution should be taken during spudcan design: for the same stiff-soft-stiff soil in situ, the spudcan peak capacity factor will be increased by increasing the spudcan size. However, at the same time, the potential of punch-through failure will be increased as well, which should be assessed.

4.6 REFERENCES


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Chapter 4: Spudcan Penetration in Stiff Uniform Clays With an Interbedded Soft Layer


CHAPTER 5 SPUDCAN PENETRATION IN A STRONG LAYER INTERBEDDED IN SOFT UNIFORM CLAYS

5.1 INTRODUCTION

Over the last decade, research on punch-through failure has been focusing on spudcan on double layer soil profiles comprising of a strong soil layer overlaying a soft soil layer (Lee, 2009, Teh et al., 2009, Hossain and Randolph, 2010, 2010, Teh et al., 2010). However multi-layered soils are prevalent in the deep water environment. Stratified soil conditions can be resulted from accelerated consolidation of certain favoured zone or other geological processes (Hooper, 1980, Castleberry and Prebaharan, 1985). A typical layered soil profile can be described as a crust layer sandwiched between overlying soft clays and underlying soft to firm clays. These crustal clay layers are typically encountered between depths of 5 to 20 m below mud line where most rig spudcans are located (Osbourne and Paisley, 2002).

In this Chapter, the effect of a thin strong soil layer sandwiched between two soft uniform clay layers on spudcan penetration responses was studied. At first, numerical analysis on spudcan penetration into three-layered uniform soil with stiff layer in the middle has been presented as a preliminary study, describing the effect of top layer on the punch-through failure potential. The thin strong layer can be either a stiff clay layer or a sand layer to understand the basic flow mechanism. The patterns of punch-through
failure and squeezing on multi-layered soils were identified. These phenomenon also follow the recent study on centrifuge modelling of spudcan foundations penetrating through multi-layered soil with interbedded strong layers (Hossain and Randolph, 2012). At the next stage, the bearing capacity profiles and associated soil flow mechanisms of a common spudcan foundation penetrating into a three layer soft-stiff-soft clay soil have been studied in detail. Both soil layer thickness and soil layer strength ratios were varied to study their effect on the spudcan penetration responses. The LDFE results of spudcan penetration into the soft-stiff-soft clay soils were calibrated by using existing centrifuge test data. A parametric study was then conducted to study the bearing capacity response and soil flow mechanisms during deep spudcan penetrations by varying the soil layer strength ratio and relative layer thickness to the diameter of spudcan.

5.2 SPUDCAN IN SOFT CLAYS WITH A THIN INTERBEDDED STIFF CLAY OR SAND LAYER

5.2.1 Numerical analysis setup and input parameters

An axisymmetric idealization has been adopted for the vertical penetration of a circular spudcan with a diameter D as illustrated in Figure 1. The spudcan was considered rough with its diameter of $D = 14$ m. A thin strong middle layer with $H_m/D = 0.25$ was interbedded. The relative thickness of the top soft clay layer varied as $H_t/D=0.5, 1, 1.5$. The soil domain was extended away from the spudcan 10 D to avoid boundary effects.
In this study all the clay layers were assumed to be undrained with uniform shear strength $s_{u1} = s_{u3} = 30$ kPa for the soft layer and $s_{u2} = 60$ kPa for the middle stiff layer. Both soil friction and dilation angles were defined as $0^\circ$ due to undrained condition and a uniform stiffness ratio $E/s_u = 500$ was selected. The saturated unit weights for both clays were taken as $\gamma_c = 17$ kN/m$^3$ and unit weight of water was $\gamma_w = 10$ kN/m$^3$. For the cases with sand layer in the middle, sand was assumed to be drained, medium dense sand with its friction angle of $32^\circ$ and its dilation angle as a small value to account for minimal dilation to maintain and keep numerical stability at the same time. The cohesion of sand layer, $c$, was assumed at a small value. The details of the soil parameters are listed in Table 1.
Table 1 Soil parameters

<table>
<thead>
<tr>
<th></th>
<th>E/MPa</th>
<th>v</th>
<th>$s_0$/kPa</th>
<th>$c'/kPa$</th>
<th>$\phi/\degree$</th>
<th>$\psi/\degree$</th>
<th>$\gamma$/kN/m$^3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft clay</td>
<td>15</td>
<td>0.49</td>
<td>30</td>
<td></td>
<td>7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stiff clay</td>
<td>30</td>
<td>0.49</td>
<td>60</td>
<td></td>
<td>7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand</td>
<td>50</td>
<td>0.3</td>
<td>0.2</td>
<td>32</td>
<td>2</td>
<td>10</td>
<td></td>
</tr>
</tbody>
</table>

Table 2 provides a summary of all cases considered for numerical analyses. Two groups of soil configurations were set up. In each soil group, there were three different relative top layer thicknesses from 0.5D to 1.5D. The tip of the spudcan was initially embedded 0.3m into the soil surface before penetration commenced as shown in Figure 2. This is to initiate a stable simulation and save some computational time. Since the tip resistance is minimal relative to the spudcan penetration response, it should not affect the overall analysis result. The penetration depth was taken relative to the bottom of the widest part of the spudcan Figure 2. Hu & Randolph (Hu and Randolph, 1998) demonstrated that accurate results can be obtained if the displacement increments are 0.1-0.2% of the foundation radius with remeshing every 10-20 steps. The dynamic time step feature of the AFENA software allowed for automatic adjustment of each small strain displacement based on the minimum element side length in the mesh. An optimal element side length to displacement ratio of 3 was selected to minimize displacement while minimizing the total number of steps required to reach sufficient penetration. A maximum displacement increment of 0.001m, i.e. 0.014% of the spudcan radius (i.e. 0.007%D), was set for each small strain time step. Remeshing of the domain occurred every 50 small strain time steps in order to minimize the amount of storage required for 5-4
the numerous simulations undertaken in this chapter. This frequency of remeshing has been used in previous studies (Yu et al., 2011, Gao et al., 2012) and still ensures data with minimal noise due to interpolation effects. Penetration proceeded until a total penetration depth of at least 3D was reached in order to obtain both peak and residual bearing capacities.

Table 2 Summary of numerical tests conducted

<table>
<thead>
<tr>
<th>Test</th>
<th>D/m</th>
<th>Top Layer</th>
<th>Middle Layer</th>
<th>Bottom Layer</th>
<th>H/D</th>
<th>Hms/D or Hms/D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group 1</td>
<td>14</td>
<td>Uniform Soft Clay</td>
<td>Medium Dense Sand</td>
<td>Uniform Soft Clay</td>
<td>0.5, 1, 1.5</td>
<td>0.25</td>
</tr>
<tr>
<td>Group 2</td>
<td>14</td>
<td>Uniform Soft Clay</td>
<td>Uniform Stiff Clay</td>
<td>Uniform Soft Clay</td>
<td>0.5, 1, 1.5</td>
<td>0.25</td>
</tr>
</tbody>
</table>

Figure 2 Initial mesh with position reference point indicated
5.2.2 Spudcan in soft clay with interbedded thin sand layer

The penetration responses of a spudcan into clay-sand-clay soil, with a thin middle sand layer \((H_m/D=0.25)\), are plotted in Figure 3. The penetration response of a spudcan into a uniform clay layer is also displayed in Figure 3 for comparison, i.e. \(H/D = \infty\). It can be seen from Figure 3 that at the beginning of penetration the vertical bearing capacities for all cases coincide with that of the uniform clay case. However after the penetration depth reaches a certain position where 0.25 D ahead of the sand layer surface, the strong middle layer begins to play a role. The vertical bearing capacities for all three-layered cases increase dramatically and peak values can be observed clearly after the spudcan foundation reaches the strong soil surface. It also can be seen that as the location of sand layer goes deeper, the peak value is clearly increasing due to high soil strength generated by deep embedded sand, besides which the punch-through failure potential is increasing as well. According to the result for \(H/D=0.5\) case, it is reasonable to say that there is no peak bearing capacity/punch-through potential when \(H/D\) is 0.5 or lower.

The deep penetration capacities for all the three-layered cases converge into the same curve and keep increasing with the penetration depth. This marks the deep penetration response where the top layer thickness is not affecting the spudcan bearing capacity anymore. The increasing trend of residual bearing capacity is due to the sand plug trapped underneath getting thicker and generating higher strength, showing in soil flow mechanisms shown in Figure 4.
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Figure 3 Load-penetration profiles for mid-sand layer (Group 1 in Table 2)

Figure 4 shows the soil flow mechanism, soil layer deformation and the sand plug for test Group 1 (clay-sand-clay, $H_t/D=1$). From the beginning of penetration (Figure 4 (a)) the top layer soil under spudcan moves downward and behaves similar to a single layer soil. At this stage the spudcan bearing capacity stays together with that of a single uniform clay case in Figure 3.

With further penetration (Figure 4 (b)), most of the soft top layer soil under the spudcan has been squeezed sideways and a shear band is formed in the sand layer. Thus the spudcan capacity has reached peak (Figure 3). At this stage, the sand layer is too strong to move upward and a large scale of rotational flow cannot be formed, hence the spudcan foundation cannot be fully embedded as in single layer clay case.

When the sand layer underneath the spudcan is forced down into bottom clay layer (Figure 4 (c)), a rapid reduction in spudcan capacity is observed in Figure 3. The bottom
soft clay soil tends to move upward around the spudcan and the sand layer is broken at the previous shear band location. Thus a soil back-flow occurs.

When the spudcan is fully penetrated into the bottom clay layer (Figure 4 (d, e)), the top clay layer is isolated at the top, forming a cavity above the spudcan, i.e. $H_c/D \approx 1$ which is the top clay thickness and is much higher compared with that in the single layered clay case, i.e. $H_c/D = 0.52$ for $s_u = 30 \text{ kPa}$ and $\gamma' = 7 \text{ kN/m}^3$ (Hossain et al., 2003, Hossain et al., 2005). During this stage, the sand flows into the cavity and fully embeds the spudcan. Meanwhile a stable sand plug underneath the spudcan is formed and is moving downward with the foundation, which enlarges the effective foundation size and produces a bearing resistance 25% higher than that of spudcan in single clay layer (Figure 3).
5.2.3 **Spudcna in soft clay with interbedded thin stiff clay layer**

The penetration responses of a spudcan into three-layered uniform clay with a stiff clay layer in the \((H_m/D=0.25, s_u = 60\text{kPa})\) are presented in Figure 5. The curve of a spudcan on a uniform clay \((H_t/D = \infty)\) is also shown in the figure for comparison.

It can be seen that the spudcan penetration responses show a pattern similar to those for clay-sand-clay cases, where at the beginning of penetration the vertical bearing
capacities for all three-layered cases coincide with that of uniform clay case. When the penetration gets deeper the spudcan foundation can sense the stiff boundary at about 0.25 D away from it, then the middle stiff clay layer starts to play a role. After that the vertical bearing capacities for all three-layered cases got increased remarkably due to the stiff clay layer and peak values are only observed for the cases with $H/D=1$ and 1.5 when the spudcan foundation almost passes through the entire stiff layer. However, the peak capacities appear later and the convergence of all three responses at deep penetration after passing the middle stiff clay, forms later than those observed for clay-sand clay soils.

For the responses in soft-stiff-soft clay soils of $H/D=0.5$, after the peak capacities are reached, the responses are kept fairly stable with some further penetration. Then a reduction in capacity (point X in Figure 5) is observed. This reduction is due to the stiff soil trapped underneath the spudcan being squeezed out in the bottom layer, which may induce a punch-through failure even there isn’t a peak capacity observed in the stiff clay layer at $H/D = 0.5$. This is also the reason why the three residual capacity curves take longer time to merge together at deep penetration. Since the most of middle stiff clay trapped underneath the spudcan has been squeezed out in the bottom layer, the residual capacity of spudcan is only slightly higher than the spudcan deep penetration capacity in a single uniform clay layer. The layer deformation is shown in Figure 6.
Once the sand layer is replaced by stiff clay layer the soil-flow mechanism becomes quite different. Figure 6 shows the soil flow mechanism and soil layer deformation for soft-stiff-soft clay soils of Group 2 (Ht/D=1). From the beginning of penetration (Figure 6 (a)) the middle strong layer doesn’t have much effect on soil behaviour and the vertical bearing capacity in Figure 5 coincides with that of the uniform clay case.

With further penetration (Figure 6 (b)), the soft top layer is squeezed sideways and the middle layer is bended significantly. A shear plane is generated in the middle stiff clay layer. The spudcan capacity has reached its peak (Figure 5). By comparing with Figure 4 (b), it is apparent that, in the soft-stiff-soft clay soil, the all three soil layers outside of the spudcan flow upwards and onto the top of the spudcan, which was not observed in the clay-sand-clay soil.

Figure 5 Load-penetration profiles for mid-clay layer (Group 2 in Table 2)
As the penetration gets deeper (Figure 6 (c)), both top soft clay and middle stiff clay layers have been squeezed away. A full rotational flow can be observed. Soil in lower layer starts to flow back to the cavity and the corresponding bearing capacity in Figure 5 tends to be stabilized. However as the penetration goes on (Figure 6 (d)), the stiff soil trapped underneath is gradually flow upwards around the spudcan due to the effect of trapped soft top layer. During this slow process, the corresponding bearing capacity (Figure 5) is reduced and slowly reached a constant value.

In the final stage, (Figure 6 (e)) unlike the clay-sand-clay case, no stiff soil is trapped under the foundation at all. A stable fully rotational soil flow which crosses through a part of stiff soil is observed around the foundation, leading to a 6% higher residual capacity compared with that in single layer case. This increased capacity may be due to the stiff soil wedge formed on top of the spudcan.
5.2.4 Summary of spudcan in soft clay with a thin stiff soil layer

For all the cases involved in preliminary study, with increasing top soft clay layer thickness, the peak capacity of spudcan is increased, followed by a reduction of capacity with further penetration. When the middle layer is stiff clay, a second capacity reduction is observed after the reduction after the peak capacity, when the stiff clay underneath
the spudcan is completely squeezed out. The cavity formed above the spudcan in the three-layer soils is much higher than that in a single clay layer. For the clay-sand-clay soil profile, a stable sand plug formed underneath provides higher strength as it goes deeper and results in an increasing residue bearing capacity 25% higher than that in a single clay layer. For the soft-stiff-soft clay soil profile, during the penetration, the stiff soil initially trapped under spudcan is gradually squeezed out and flows outwards and upward. This process leads to a secondary bearing capacity reduction after punch-through failure. The residue capacity for three-layered case is eventually stable and 6% higher than that in single layer case.

Crucially based on the understanding on the soil flow mechanisms from this study, the combined soil flow effects on the bearing capacity factor \( N_c \) for spudcan penetrating into soft-stiff-soft profile can be summarized in Figure 7. Also in order to predict the \( N_c \) profile accurately, nine factors listed below should be fully understood.

(1) When the penetration starts, a transition from shallow heave to deep flow around foundation may occur where the \( N_c \) profile starts to climb up from \( N_{surface} \).

(2) Afterwards a squeezing flow will be triggered between the stiff layer and spudcan, leading a dramatic increase on \( N_c \).

(3) When the foundation punching through the stiff layer a direct shear plane on the stiff layer will be identified.

(4)/(5) The \( N_c \) increase due to the contribution of stiff soil layer while the stiff layer thickness is decreasing resulting in a reduction on \( N_c \).
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(6) When the soil starts to flow back into the cavity after punching through the stiff layer and losing overburden effect, a reduction on $N_c$ can be observed.

(7) The stiff plug underneath the spudcan will be minified with further penetration.

(8) Part of the plug will diminish.

(9) Part of the plug will be stabilized underneath the spudcan and act as part of the foundation.

This study investigated spudcan foundations penetrating into three layer uniform clays; identified soil flow mechanisms associated with those factors and to quantify their effect on the $N_c$ profile, which was calculated using soft clay strength.

![Figure 7 Bearing factor $N_c$ profile prediction for spudcan penetration on three layered soil with a stiff layer in the middle](image)

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5.3 PARAMETRIC STUDY ON SPUDCAN PENETRATING INTO UNIFORM CLAYS WITH A STRONG LAYER IN THE MIDDLE

5.3.1 Numerical analysis setup and input parameters

For the parametric study a similar axisymmetric idealization has been adopted for the vertical penetration of a circular spudcan with a diameter D as illustrated in Figure 8. Spudcan was considered smooth (as a comparison with rough base cases above in section 5.2) with its diameter of D = 14 m. A relatively stiff middle layer with H2/D = 0, 0.25, 0.5, 1, 1.5 was interbedded. The relative thickness of the top soft clay layer varied as H1/D=0, 0.25, 0.5, 0.75. The soil domain was set up 10D by 10D to avoid boundary effect.

In this study all the clay layers were assumed to be undrained with uniform shear strength $s_{u1} = s_{u3} = 10, 20$ kPa for the soft layer and $s_{u2} = 40, 60, 80$ kPa for the middle stiff layer. Both soil friction and dilation angles were defined as 0° due to undrained...
condition and a uniform stiffness ratio $E/s_u = 500$ was selected. The saturated unit weights of clays were both taken as $\gamma' = 7\text{kN/m}^3$ and water unit weight was $\gamma_w = 10\text{kN/m}^3$. The details of the soil parameters are listed in Table 3.

<table>
<thead>
<tr>
<th>Group NO.</th>
<th>Soil strength ratio ($s_{u1}/s_{u2}$)</th>
<th>$s_{u1}$ (kPa)</th>
<th>$s_{u2}$ (kPa)</th>
<th>$H_1/D$</th>
<th>$H_2/D$</th>
<th>$D$ (m)</th>
<th>$s_{u1}/\gamma' D$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>0.5</td>
<td>20</td>
<td>40</td>
<td>0,0.25,0.5,0.75</td>
<td>0,0.25,0.5,1,1.5</td>
<td>14</td>
<td>0.204</td>
</tr>
<tr>
<td>II</td>
<td>0.33</td>
<td>20</td>
<td>60</td>
<td>0,0.25,0.5,0.75</td>
<td>0,0.25,0.5,1,1.5</td>
<td>14</td>
<td>0.204</td>
</tr>
<tr>
<td>III</td>
<td>0.25</td>
<td>20</td>
<td>80</td>
<td>0,0.25,0.5,0.75</td>
<td>0,0.25,0.5,1,1.5</td>
<td>14</td>
<td>0.204</td>
</tr>
<tr>
<td>IV</td>
<td>0.25</td>
<td>10</td>
<td>40</td>
<td>0,0.25,0.5</td>
<td>0,0.5,1</td>
<td>14</td>
<td>0.102</td>
</tr>
</tbody>
</table>

5.3.2 Effect of top layer thickness $H_1/D$

The bearing capacity profiles of a spudcan penetrating into soil with relatively stiff layer thickness $H_2/D = 1$, for soil strength ratio $s_{u1}/s_{u2} = 0.33$, are shown in Figure 9.

It can be seen that the initial penetration profile of the three-layer cases follows the trend of the single layer case. The profiles begin to deviate at a distance of approximately 0.4D above the stiff layer, where the stiff layer begins to have an influence. This process was controlled by the squeezing soil flow (Figure 7). The rates of increasing capacity with different $H_1/D$ are similar. For cases $H_1/D = 0, 0.25, 0.75$ during foundation penetrating through the stiff layer, the increasing $N_c$ clearly outweighs the stiff layer thinning effect. This results in a constant rate increase in bearing capacity gradient throughout the stiff layer penetration until a peak is reached when the soil starts to flow back losing overburden effect. Most interestingly, the $H_1/D$ 5-17
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= 0.5 profile saw a peak in bearing capacity while less than half way into the stiff layer, after initially increasing in capacity at a greater rate. A reduction in bearing capacity occurs at this point, and the bearing response then follows in a trend similar to the previous two cases before another peak capacity 0.15D below the stiff layer. This phenomenon is linked to the soil flow mechanisms discussed below.

A series of soil flow pictures for this case were cut out along the penetration at different depths in Figure 9. With initial penetration, the top soft clay layer under the spudcan is displaced downwards. Heaving of the top layer eventually occurs, due to the lateral squeezing of soil from beneath the foundation base as the middle layer acts as a stiff boundary. The degree of squeezing is sufficient to initiate flow of the soft clay onto the spudcan surface (Figure 9 1). As the foundation enters deeply into the stiff layer, the soft soil sitting on spudcan displaces downwards with it. No soft top layer material trapped underneath at this point due to the smooth base of spudcan. This is different from the rough base cases (Gao et al., 2012) where the soft top layer material was trapped underneath the spudcan during its penetration into the stiff layer. The amount of soil on top of the spudcan gradually decreases as penetration continues until no more remains (Figure 9 2 and 3). This process where the top layer flows back was interrupted by penetration into stiff layer corresponds to the reduction in bearing capacity witnessed in the Nc profile at approximately d/D = 0.87. The soil has sufficient strength to adhere to the vertical cavity wall of the stiff layer and continue being lumped over the edge for remainder of the penetration (Figure 9 4). After that the second layer started to flow back to the cavity and the punch-through failure can be observed (Figure 9 5 and 6). This phenomenon is not present in other three cases as explained below.
Figure 9 ((a) to (d)) illustrates the soil flow mechanisms at peak penetration resistance at 0.2D below the stiff layer, which are marked by solid circles on Nc profiles. For all cases, except H1/D = 0.5, there is no interruption of soil back flow observed, where soil cavity is kept forming for H1/D = 0 and 0.25 or soil back flow is continuing for H1/D = 0.75. Thus, there is no initial peak observed when spudcan enters the stiff layer. For H1/D = 0.25, no soil backflow occurs due to squeezing of the top layer. Thus stiff layer dominates cavity formation. When H1/D = 0.5, top soil briefly fills the cavity during penetration into the stiff layer, before adhering to vertical cavity wall of stiff clay. The cavity depth predicted by (Hossain et al., 2006) can be expressed as:

\[
\frac{H_c}{D} = \left( \frac{\sigma_u H}{\gamma D} \right)^{0.55} - \frac{1}{4} \frac{\sigma_u}{\gamma'} D
\]  

This equation predicts a backflow of top layer soil to occur at a depth of 0.366D. However an extra penetration distance is still needed to develop a full back flow. Since 

\[
\left( \frac{\sigma_u H}{\gamma D} \right)^{0.55} - \frac{1}{4} \frac{\sigma_u}{\gamma'} D < H_1/D = 0.5 < \left( \frac{\sigma_u H}{\gamma D} \right)^{0.55} - \frac{1}{4} \frac{\sigma_u}{\gamma'} D
\]

and H2/D is relatively thick and stiff, the first layer inflow is stopped by the punching through effects and the second layer soil will flow back to the cavity eventually during further penetration. When H1/D is increased to 0.75, the shear strength of the upper clay is no longer sufficient to prevent top soil collapse, a full rotational flow was developed within the top layer and the clay partially fills the cavity during further penetration.

The degree to which the stiff layer soil near the top layer heaves upwards during this rotational soil flow is reduced as H1 increases (Figure 9 (a)--(d)). This is attributed to the additional weight of overburden confining the stiff layer.

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After the soil flows into the cavity and develops full rotational flow around the foundation, the residual capacities of all cases displayed in Figure 9 show a convergence to the lower bound $N_c$ profile with a similar reducing rate. This indicates that the spudcan plug size was minimized until nothing left underneath and this process is governed mainly by the strength ratio $s_u1/s_u2$. 
Figure 9 Effect of $H_1$ on bearing capacity profiles for Group II \((S_{u1}/S_{u2}=0.33, S_{u1}/\gamma' D=0.204), H_2/D =1\)
Figure 10 shows the bearing capacity for the Group II cases with H₂/D = 0.25 in Table 3. By comparing Figure 9 and Figure 10, the effect of H₁/D for a smaller H₂/D can be observed. In Figure 10, the squeezing effect was not as severe as that in Figure 9 since a thinner stiff layer is much easier to bend. Both the stiff-over soft case and H₂/D = 0.25 case exhibit a small degree of punch through while in the stiff layer, followed by similar rises in bearing capacity before reduction to a residual value. The profile corresponding to H₁/D = 0.5 does not punch through until passing fully through the thin middle layer, however a large reduction occurs after peak which was not seen for the equivalent cases with thinner top layers. For the case with H₁/D =0.75, the ultimate capacity is somewhat larger than what the trend would assume, and the largest post peak reduction is observed.
Chapter 5: Spudcan Penetration in a Strong Layar Interbedded in Soft Uniform Clays

Figure 10 Effect of $H_1$ on bearing capacity profiles for Group II ($s_{u1}$

$/s_{u2}=0.33$, $s_{u1}/\gamma'D=0.204$), $H_2/D=0.25$

By comparing Figure 11 and Figure 12 the effect of $H_1/D$ for a smaller $s_{u2}/\gamma'D$ can be observed. Figure 11 displays the bearing capacity profiles for the Group III cases ($s_{u1}

$/s_{u2}=0.25$, $s_{u1}/\gamma'D=0.204$), for $H_2/D = 1$ and Figure 12 shows the Group III cases ($s_{u1}

$/s_{u2}=0.25$, $s_{u1}/\gamma'D=0.102$), for $H_2/D = 1$. In group IV $s_{u1}$ was reduced from 20 to 10kPa.

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The capacity profiles in Figure 11 follow a path completely different from the equivalent case in Figure 12. Much higher peak positions are observed in Figure 8, indicating for cases in Figure 11 after punching into the stiff layer, the stiff plug thinning effect outweighs the increasing $N_c$ listed in Figure 7. In Figure 12 the soil flow mechanisms at the bottom of stiff layer are also displayed. Figure 12 (b) shows the top layer flow back process is interrupted when $H_1/D = 0.25$ in group IV and a minor reduction in bearing capacity is apparent at $\sim 0.25D$ above the bottom layer interface. When $H_1/D = 0.5$, the capacity gradient actually increases with penetrating into the stiff layer. However with lower $s_{u1}/\gamma' D = 0.102$ the top layer is more easily flowing into the cavity which reduces the overburden effect. Figure 12 (c) shows similar $N_c$ profile compared to case $H_1/D = 0.25$. The residual capacities of both group III and group IV multi-layer cases do not converge with the single case due to the presence of a soil plug beneath the spudcan with identical strength ratio $s_{u1}/s_{u2}$.
Figure 11 Effect of $H_1$ on bearing capacity profiles for Group III ($s_{u1}/s_{u2}=0.25$, $s_{u1}/\gamma' D=0.204$), $H_2/D=1$
Figure 12 Effect of $H_1$ on bearing capacity profiles for Group IV ($s_{u1}/s_{u2}=0.25$, $s_{u1}/\gamma' D=0.102$), $H_2/D =1$
5.3.3 Effect of strength ratio $s_{u1}/s_{u2}$

The effects of soil strength ratio $s_{u1}/s_{u2}$ and relative soft layer strength $s_{u1}/\gamma'D$ are displayed in Figure 13. It is clear that the ultimate bearing capacity factor is increasing with decreasing strength ratio, as the strength of the middle layer increases. Before the foundation penetrating into the stiff layer different degrees of squeezing for cases $s_{u1}/s_{u2}=0.5, 0.33, 0.25$ can be observed where 2 cases are with same strength ratio of 0.25.

A premature reduction is seen for the $s_{u1}/s_{u2}=0.33, 0.25$ ($s_{u1}/\gamma'D =0.204$) due to the interruption of top soil back flow. The depth of capacity peak below the stiff layer increases with decreasing strength ratio. This is due to a stiffer middle layer more resistant to rotational soil flow into the cavity.

Figure 13 ((a)—(d)) illustrates how the increase in the soil strength ratio and $s_{u1}/\gamma'D$ influences the soil flow mechanisms during penetration, which corresponds to the bearing capacity profile in Figure 13 at the penetration depth of $d/D=2$. It is clear that with the increasing soil strength ratio the top layer flow mechanism was changing from sideway squeezing to a critical state where the top layer adhering to vertical cavity wall Figure 13 ((a)-(c)). Comparing Figure 13 (d) with Figure 13(c) it can be seen that with low $s_{u}/\gamma'D$ a full rotational flow was developed earlier and the higher $N_c$ was achieved.

The residual capacities converged in different ways due to different strength ratio $s_{u1}/s_{u2}$. By observing the Figure 13 ((a)—(d)) it’s also clear to see the strength ratio controls the plug size underneath the spudcan. Figure 14 also presents the spudcan penetration responses and show mainly two types of peaks: one happens in the top of stiff layer, another one happens in the bottom or even third layer, depends on when full circulated soil flow could be developed around the spudcan brim. The peak resistance is
always the main interest for design. The peak value and the location when the peak occurs could be estimated as shown below:

For all the cases conducted, only when strength ratio $s_{u1}/s_{u2} \leq 0.25$  $s_{u1}/\gamma' D \geq 0.204$ and stiff layer $H_2/D \geq 1$. A peak occurs at the top of the stiff layer

Peak location:

$$\frac{d_{peak}}{D} = 1.5 \left[ \left( \frac{s_{u3}}{s_{u2}} \right)^a \left( \frac{H_1+H_2}{D} \right) \right]$$  (2)

a=1.1

For majority of the cases, when strength ratio $s_{u1}/s_{u2} > 0.25$, $s_{u1}/\gamma' D < 0.204$ and stiff layer $H_2/D < 1$. a peak occurs at the bottom of the stiff layer or even below the stiff layer

Peak location:

$$\frac{d_{peak}}{D} = 1.5 \left[ \left( \frac{s_{u3}}{s_{u2}} \right)^a \left( \frac{H_1+H_2}{D} \right) \right]$$  (3)

a=0.3

Once $d_{peak}/D$ is determined, the corresponding penetration resistance at the peak can be calculated by the following equation:

$$\frac{q_{peak}}{s_{u1}} = 5.1 + 3 \left( \frac{s_{u2}}{s_{u1}} \right) \left( \frac{H_2}{D} \right) + 3 \left( \frac{s_{u3}}{s_{u1}} \right)$$  (4)
Figure 13: Effect of $s_{u1}/s_{u2}$ on bearing capacity profiles for $H_1/D = 0.5$, $H_2/D = 1$
Spudcan foundation, as the most common shape of foundation for jack-up mobile rigs, penetrating into three-layered uniform clay with a stronger layer in the middle has been studied using LDFE/RITSS analysis.

It was found that there were three types of bearing responses during continuous penetration of spudcans: (a) when the top soft layer is relatively thin, the spudcan bearing response was similar to that of two layer soils with stiff over soft clays; (b) when the top soil layer thickness is medium, a peak resistance is observed when spudcan penetrates into the middle stiff layer followed by reduction; (c) when the soil layer is thick, the peak resistance occurs when spudcan gets into the bottom soft soil layer. The critical thickness of top soil layer is a function of soil strength ratio and middle stiff soil layer thickness. The bearing response types were also corresponding to the soil cavity formations during initial penetration of the spudcan.

Eventually the combined soil flow effects on the bearing capacity factor $N_c$ for spudcan penetrating into soft-stiff-soft profile were well studied and summarized at different penetration depth.

Different soil flow mechanisms associated with main factors (e.g. soil layer thickness, soil strength ratio etc.) listed in Figure 7 and their effect on the $N_c$ profile was investigated in detail. Research presented in this chapter establishes a solid basis for the following studies in Chapters 6 and 7 where a thin layer is sandwiched in NC clay profiles.
Figure 14: Conclusion on bearing factor $N_c$ profile prediction for spudcan penetration on three layered soil with a stiff layer in the middle.

**Conclusion**

1. $H_1/D$ needs to be much higher than $(\frac{s_{u1}}{\gamma D})^{0.55}$ to develop full rotational flow in the top clay layer.

2. The squeezing mechanism was observed at 0.4D above stiff layer for most of the cases and the degree of squeezing is mainly determined by the strength ratio of $s_{u1}/s_{u2}$ and the middle layer thickness of $H_2/D$.

3. Punch-through failure will always happen at some point for soft-stiff-soft uniform clay profiles due to the soft bottom layer.
5.4 REFERENCES


Lee, K. K. (2009). Investigation of potential spudcan punch-through failure on sand overlying clay soils, University of Western Australia Perth.
CHAPTER 6  SPUDCAN PENETRATION IN NC CLAY
WITH AN INTERBEDDED STIFF CLAY

6.1 INTRODUCTION

Since the first mobile offshore platform with spudcan foundations -Zepata’s ‘Scorpion’ started travelling between well sites in Gulf of Mexico in 1956, jack-up use has continued to be extended to deeper waters and in harsher environments (Gao et al., 2012). However the increased number of mobile jack-up rig operations have led to increased number of incidents/failures in recent years. It has been reported that unexpected punch-through failures have resulted in both rig damage and lost drilling time at a rate of 1 incident per annum with consequential costs estimated at between US$1 and US$10 million (Hossain et al., 2004). According to the previous study (Osborne et al., 2009), highly layered soil profiles are prevalent in quite a few fields, such as the Sunda Shelf, offshore Malaysia and offshore Thailand and the latter in the North Sea, offshore India, offshore Australia and in the Arabian Gulf. A case study about soil data at Yolla, offshore Australia, shows that several stiffer calcareous layers were embedded at various depths (Young et al., 1984, Hunt and Marsh, 2004).

Over the last decade, the research on spudcan foundations has focused on installing and preloading a jack-up rig on soils with strong layer overlaying soft layers (Jack et al., 2001, Kvitrud et al., 2001, Teh et al., 2008, Teh et al., 2009, Gao et al., 2012, Hossain,
Chapter 6: Spudcan Penetration in NC clay with an interbedded stiff clay

(2014), where the punch-through failure remains challenging. Although the potential hazard of unpredicted penetration incidents has been documented by various reports and proposed algorithms in design guidelines (SNAME, 2008, Osborne et al., 2009, Xie et al., 2010, Gao et al., 2012), frequent jack-up rig failures are still occurring globally till today. When multi-layered soils are encountered, the soil flow mechanism is much more complicated and the penetration responses of spudcan foundations can be more difficult to predict. The behaviour of spudcan penetrated into sand-clay-sand was investigated by Kellezi and co-workers (Kellezi and Stromann, 2003, Kellezi et al., 2008), they used large deformation FE methods provided by ABAQUS standard (Hibbitt and Sorensen, 2000) and Explicit (Hibbitt and Sorensen, 2000), and ELFEN FEM programs. Hossain and Randolph (ISO19905-1, 2012, Hossain, 2014) also reported centrifuge modelling of spudcan foundations penetrating through three-layered soil and four-layered soil profiles.

Stratified soil conditions can result from accelerated consolidation of certain favoured zones or other geological processes (Hansen, 1970, Wong et al., 2012). A typical layered soil profile can be described as a crust layer sandwiched between overlying soft clays and underlying soft to firm clays. These crustal clay layers are typically encountered between depths of 5 to 20 m below mud line where most rig spudcans are located (Hossain et al., 2004). An example is shown in Figure 1 describing the two different seabed locations in Mississippi delta (Wong et al., 2012). Recently centrifuge modelling of spudcan foundations penetrating through multi-layered soil with interbedded strong layers has been studied (Xie et al., 2010), identifying the patterns of punch-through failure and squeezing on multi-layered soils. Numerical analysis on spudcan penetration into three-layered uniform soils with a stiff layer in the middle has
also been reported (Hoyle et al., 2006) describing the effect of top layer on punch-through failure potential.

![Graph](image)

**Figure 1 Soil profiles at Mississippi delta sites (Hooper, 1980)**

To enhance the understanding of the soil failure mechanism and bearing capacity on multi-layered ocean floor, this chapter presents the penetration process of a spudcan foundation on normally consolidated (NC) clay with an interbedded stiff clay layer using LDFE/RITSS analysis.

### 6.2 NUMERICAL ANALYSIS SETUP AND INPUT PARAMETERS

#### 6.2.1. Geometry and parameters

The finite element calculations were carried out by AFENA program (Carter and Balaam, 2000). All of the FE calculations were based on six-triangular elements with three-point Gauss integration rules to calculate the element stiffness matrices. An 6-3

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axisymmetric idealization has been adopted for the vertical penetration of a circular spudcan with a diameter D as illustrated in Figure 2. The spudcan base was considered both rough and smooth with its diameter of D = 12 m. A stiff middle layer with H2/D = 0.25, 0.5, 1 was interbedded in an NC clay layer. The relative thickness of the top soft clay layer varied as Hf/D=0, 0.5, 1, 1.5. The soil domain was set up 10 D by 10 D to avoid boundary effect.

![Figure 2 Schematic diagram of spudcan penetration in three-layered clay](image)

In this study the clay layers in all cases were assumed to be undrained. The undrained shear strength for NC clay was described as su1=sa3=kxz kPa as shown in Figure 2 with k = 1.0 and 1.5 kPa/m as a typical undrained shear strength gradient lies within the range of k = 0 ~ 2 kPa/m in deep water (Lee, 2009). The undrained shear strength of the middle stiff clay is s_u2, which is referred to as s_u stiff in Table 1. The soil strength ratio of the stiff clay layer to the bottom layer at the top interface of \( \frac{s_{u\text{stiff}}}{k(H_1+H_2)} \) varied from 2 to 6 to study its effect on the punch-through failure. Both soil friction and dilation angles were defined as 0° due to undrained condition and a uniform stiffness ratio E/s_u = 500.
was selected. The saturated unit weights of clays were both taken as $\gamma_e = 16\text{kN/m}^3$ and water unit weight was $\gamma_w = 10\text{kN/m}^3$. The details of the soil parameters are listed in Table 1.

Table 1 Summary of LDFE analysis of spudcan on NC clay with interbedded stiff layer

<table>
<thead>
<tr>
<th>Spudcan base roughness</th>
<th>$\alpha$</th>
<th>0</th>
<th>0.5</th>
<th>1</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normalised top layer thickness</td>
<td>$H_t/D$</td>
<td>0</td>
<td>0.5</td>
<td>1</td>
<td>The penetration starts from the widest brim touches the seafloor as shown in Figure 2.</td>
</tr>
<tr>
<td>Normalised middle layer thickness</td>
<td>$H_m/D$</td>
<td>0.25</td>
<td>0.5</td>
<td>1.0</td>
<td>LDEF analyses investigate the effect of interface friction, normalised layer thickness, shear strength gradient and strength ratio.</td>
</tr>
<tr>
<td>Shear strength gradient</td>
<td>$k$ (kPa/m)</td>
<td>1</td>
<td>1.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strength ratio of stiff middle layer and the surface of bottom layer</td>
<td>$s_{u, stiff}/k(H_t + H_m)$</td>
<td>2</td>
<td>4</td>
<td>6</td>
<td></td>
</tr>
</tbody>
</table>

Table 2 Summary of LDFE analysis in study groups

<table>
<thead>
<tr>
<th>Group Number</th>
<th>$H_t/D$</th>
<th>$H_m/D$</th>
<th>$k$ (kPa/m)</th>
<th>$s_{u, stiff}/k(H_t + H_m)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group I-1(a,b)</td>
<td>0</td>
<td>0.25, 0.5, 1.0</td>
<td>1(a), 1.5(b)</td>
<td>2</td>
</tr>
<tr>
<td>Group I-2(a,b)</td>
<td>0</td>
<td>0.25, 0.5, 1.0</td>
<td>1(a), 1.5(b)</td>
<td>4</td>
</tr>
<tr>
<td>Group I-3(a,b)</td>
<td>0</td>
<td>0.25, 0.5, 1.0</td>
<td>1(a), 1.5(b)</td>
<td>6</td>
</tr>
<tr>
<td>Group II-1(a,b)</td>
<td>0.5</td>
<td>0.25, 0.5, 1.0</td>
<td>1(a), 1.5(b)</td>
<td>2</td>
</tr>
<tr>
<td>Group II-2(a,b)</td>
<td>0.5</td>
<td>0.25, 0.5, 1.0</td>
<td>1(a), 1.5(b)</td>
<td>4</td>
</tr>
<tr>
<td>Group II-3(a,b)</td>
<td>0.5</td>
<td>0.25, 0.5, 1.0</td>
<td>1(a), 1.5(b)</td>
<td>6</td>
</tr>
<tr>
<td>Group III-1(a,b)</td>
<td>1</td>
<td>0.25, 0.5, 1.0</td>
<td>1(a), 1.5(b)</td>
<td>2</td>
</tr>
<tr>
<td>Group III-2(a,b)</td>
<td>1</td>
<td>0.25, 0.5, 1.0</td>
<td>1(a), 1.5(b)</td>
<td>4</td>
</tr>
<tr>
<td>Group III-3(a,b)</td>
<td>1</td>
<td>0.25, 0.5, 1.0</td>
<td>1(a), 1.5(b)</td>
<td>0, 0.1</td>
</tr>
</tbody>
</table>

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6.2.2. Mesh density and displacement increment study

A set of preliminary calculations for spudcan foundation on normally consolidated (NC) clay ($s_u = 1.5z$ kPa) was carried out to study the solution dependency on displacement increment (DT) and the mesh density. The optimal mesh is generated by refining the elements close to the foundation. A typical mesh for spudcan foundation is shown in Figure 3. The penetration starts from the widest brim of foundation touching the seafloor.

![Typical initial mesh for spudcan foundation with h_{min}/D=0.017](image)

6.2.3. Comparison with existing data

In order to calibrate current FE analysis results, the bearing capacity factor $N_c$ of spudcan penetrating into NC clay deposit was compared with exiting results in Figure 4. It can be seen that the ultimate bearing capacity factor from current FE analysis is $N_c = 10.3$ which is not much different from $N_c = 10.5$ in centrifuge test and $N_c = 10.1$ in FE analysis by (Hossain et al., 2011). Also they mentioned that the centrifuge results lay between the results of smooth and rough spudcan from FE analysis. Although the
spudcan surface in the physical test was polished, it might not be perfectly smooth which could result in a slightly higher $N_c$ number.

![Figure 4 Bearing capacity factor, $N_c$ of spudcan in NC clay](image)

Meanwhile the spudcan load penetration responses into multi-layered soil profile between current FE analysis and previous centrifuge test (Hossain et al., 2011) were compared in Figure 5. It can be seen that the FE result shows almost the same vertical bearing pressure along the penetration and similar peak position as the centrifuge test result. The bearing pressure initially increases gradually in the top layer of NC clay but grows dramatically as the foundation approaches the middle stiff layer. The FE result shows the same squeezing effect as the physical test due to the relative thickness of stiffer layer and the strength of upper layer and lower layer.
6.3 SOIL LAYER DEFORMATION AND FLOW MECHANISM - DOUBLE-LAYER VS THREE-LAYER SOILS

Typical soil flow mechanisms during penetration of a spudcan in a three-layered soil with strength ratio of \( \frac{\beta_{\text{stiff}}}{k(H_t+H_m)} = 6 \) (\( \alpha = 0 \), Group III-3(b) \( H_t/D = 1 \), \( H_m/D = 0.25 \) in Table 2) are displayed in Figure 6. The soil flow mechanisms for a severe punch-through failure obtained for three-layer cases (the corresponding bearing pressure \( q_u \) profile can
be found in Figure 9 for further discussion) and a double-layer case with the same strength ratio $\frac{s_{\text{stiff}}}{k(H_t+H_m)}$ was also present as below for comparison ($\alpha = 0$, Group I-3(b) $H_t/D = 0$, $H_m/D = 0.25$ in Table 2).

<table>
<thead>
<tr>
<th>Cases</th>
<th>Squeezing mechanism finishing</th>
<th>Punch-through</th>
<th>Plug forming</th>
<th>Plug sliding</th>
<th>Upside plug</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H_t/D=1$</td>
<td>$H_m/D=0.25$</td>
<td>$k=1.5kPa/m$</td>
<td>$s_{\text{stiff}}/k(H_t+H_m)=6$</td>
<td>12133</td>
<td></td>
</tr>
<tr>
<td>(Comparison)</td>
<td>$H_t/D=0$</td>
<td>$H_m/D=0.25$</td>
<td>$k=1.5kPa/m$</td>
<td>$s_{\text{stiff}}/k(H_t+H_m)=6$</td>
<td>10133</td>
</tr>
</tbody>
</table>

Figure 6 Soil failure mechanisms for a smooth spudcan in three and two layered clays ($\alpha = 0$, $H_m/D = 0.25$, Group III-3(b), $H_t/D = 1$ and Group I-3(b) $H_t/D = 0$ in Table 2)

The design algorithm in the guidelines (SNAME, 2008, Xie et al., 2010) is based on the solutions of foundation in single and two-layer soils which have been developed by previous researchers (Hossain and Randolph, 2009, Teh et al., 2009). However from the results in Figure 6, a significantly different flow mechanism can be identified in a three-layer soil profile (i.e a stiff layer interbedded in a NC layer) compared to double layered case (i.e. a stiff clay layer over a soft NC clay layer). In the three-layer case with a
relatively soft NC clay layer on the top, it is very easy for the top soft soil to flow back into the cavity above the spudcan and to fill up the cavity completely. The back-flow soil can confine the movement of middle stiff layer during spudcan penetrating through it. This confinement results in more stiff soil trapped underneath the spudcan as a larger plug moving downward with the spudcan. Also in Figure 6, a significant squeezing mechanism is observed when the spudcan is moving closer to the middle stiff layer. Therefore a new flow mechanism based design algorithm for multiple soil layer profiles needs to be established.

6.4 EFFECT OF THE STRENGTH RATIO OF STIFF MIDDLE LAYER AND THE SOIL UNDERNEATH $\frac{S_{\text{stiff}}}{k(H_t+H_m)}$

In order to demonstrate the effect of the strength ratio of the middle stiff layer to the soil underneath it ($\frac{S_{\text{stiff}}}{k(H_t+H_m)}$), bearing pressure $q_u$ profiles are plotted in Figure 7. The strength ratio is at the range of $\frac{S_{\text{stiff}}}{k(H_t+H_m)}=2, 4, 6$ with identical layer geometry and strength gradient respectively of $H_t/D=0.5, H_m/D=0.5, k=1.5$ kPa /m ($\alpha=0$, Group II-1(b) and Group II-3(b) in Table 2). The corresponding soil flow mechanism throughout the penetration depth is presented in Figure 8.

It can be seen from Figure 7 and Figure 8 that the strength ratio $\frac{S_{\text{stiff}}}{k(H_t+H_m)}$ plays an essential role on the both bearing pressure $q_u$ and soil flow mechanism. Compared to low strength of the top soil layer, for all the cases the second layer acts as a stiff boundary and a significant squeezing mechanism can be observed in Figure 8 corresponding to a dramatic increase in bearing pressure $q_u$ profile in Figure 7. However with stiffer middle layer, less associated boundary deformation can be found and less
top layer soil was trapped underneath when squeezing mechanism ends, leading to a much higher bearing pressure $q_u$ in Figure 7 (Meyerhof and Chaplin, 1953).

The potential for punch-through failure increases with increasing $\frac{s_{\text{stiff}}}{k(H_t+H_m)}$. The increasing punch-through severity with higher strength ratio is caused by a change in the combined contribution of the corresponding shear failure plane which passes through both the stiff soil and bottom NC layer, as demonstrated in Figure 8. Consequently, with higher $\frac{s_{\text{stiff}}}{k(H_t+H_m)}$, more bearing capacity will be lost when the failure plane extends from the stiff layer to the bottom NC layer and a significant reduction can be observed when $\frac{s_{\text{stiff}}}{k(H_t+H_m)} = 4.6$ in Figure 7.

During the punching through process, significant amount of stiff soil has been cut off from the middle layer by spudcan and a much larger stiff soil plug was formed under spudcan for cases with higher strength ratio $\frac{s_{\text{stiff}}}{k(H_t+H_m)}$ as demonstrated in Figure 8. This is caused by a change in the corresponding soil flow mechanisms from partial punching shear failure to full punching shear failure.

With further penetration, for all the cases displayed the plug thickness deduced with certain rates until it was left above spudcan. It can be seen from the bearing pressure $q_u$ profile in Figure 7 that for cases with lower strength ratio e.g. $\frac{s_{\text{stiff}}}{k(H_t+H_m)} = 2.4$ the bearing pressure picks up immediately after punching through the middle stiff layer as the bottom layer soil strength becomes comparable to the plug soil strength much sooner than the $\frac{s_{\text{stiff}}}{k(H_t+H_m)} = 6$ case. Low $\frac{s_{\text{stiff}}}{k(H_t+H_m)}$ results in low strength ratio of stiff plug and local soil flow around spudcan $\frac{s_{\text{stiff}}}{s_{\text{local}}}$, leading to high reduction rate on the plug thickness. Consequently as the plug minifying, the shear failure plane around foundation changes

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gradually. Especially when the plug transfers from the bottom of spudcan to the top of it, a sudden change in the corresponding soil failure plane can be observe in Figure 8 meanwhile a second bearing pressure peak can be found followed by a sudden reduction in Figure 7.

In the final stage, the stiff plug is left above the foundation and the residual bearing pressure goes back to the one layer NC clay $q_u$ profile as predicted. However for cases with higher $\frac{s_{\text{stiff}}}{k(H_t+H_m)}$, after the peak resistance it takes longer penetration depth to reach the single NC clay curve, since with higher strength ratio, it needs more soil depth to reach the soil depth with equivalent soil strength to $s_{\text{stiff}}$ for the stiff soil plug under spudcan to be diminish.

![Figure 7 Effect of strength ratio $\frac{s_{\text{stiff}}}{k(H_t+H_m)}$ on vertical bearing pressure for $\alpha= 0$, $k = 1.5$ kPa/m, Group II-1(b), Group II-2(b) and Group II-3(b) in Table 2)](image)

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### 6.5 EFFECT OF THE TOP LAYER THICKNESS RELATIVE TO THE SPUDCAN DIAMETER (H_t/D)

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In order to demonstrate the effect of the top layer thickness relative to the spudcan diameter (H_t/D) bearing pressure q_u profiles are plotted in Figure 9, for a range of H_t/D = 0, 0.5, 1 with identical soil properties and middle layer thickness respectively of H_m/D = 0.25, s_{ustiff}/k(H_t+H_m) = 6, k = 1.5 kPa/m (α = 0, Group I-3(b), Group II-3(b) and Group III-3(b) in Table 2). When H_t/D = 0, it means the soil profile is double layered soil. The corresponding soil flow mechanism throughout the penetration depth is also presented in Figure 11.

Figure 9 Effect of top layer thickness H_t on vertical bearing pressure (α= 0, k=1.5 kPa/m, s_{ustiff}/k(H_t+H_m) = 6, Group I-3(b), Group II-3(b) and Group III-3(b) in Table 2)
It can be seen from Figure 9 that with a soft NC clay layer lying above the stiff clay, a dramatic difference on bearing pressure q_u profile can be observed. The potential for punch-through failure increases remarkably with the increasing H_t/D and peak (marked with solid round dots on the curves in Figure 9 ) occurs at deeper position as H_t/D increasing. It is clear to be seen in Figure 11 that when spudcan approaches the stiff layer a significant squeezing occurs between spudcan and the stiff layer. The stiff layer was pushed downward by the top layer soil considerably for case (Smooth) Group III-3(b) H_m/D=0.25. This phenomenon follows the results from the recent experimental tests (Hossain, 2014). The likelihood and severity of stiff layer moving downward during squeezing phase is mainly dominated by the stiff layer thickness and the strength difference between the top two clay layers. In this study,

\[
\frac{s_{\text{ult}}}{s_{\text{stiff}}} = \frac{k \times H_t}{k(H_t+H_m)(H_t+H_m)} = \frac{H_t}{(H_t+H_m)(H_t+H_m)} = \frac{1}{(1+H_m/H_t)(H_t+H_m)}
\]

(\(s_{\text{stiff}}=6, H_m=0.25D\) for cases in Figure 9 )

Therefore thicker top layer will result in comparable strengths between the top two layers and soil flow mechanism at this stage can change from pure squeezing to a combination of partial squeezing and general shear failure. With the stiff layer slightly bended, the punching shear on stiff layer is delayed and leading to a peak bearing pressure happened at deeper position.

A much sever punch-through failure is observed in cases with the high H_t/D in Figure 9 leading to much more rapid bearing pressure reduction rate after the peak. It is clear from Figure 10 that the increasing top layer thickness will result in a longer penetration depth to catch up the same soil strength as the middle soil layer (15m 45mand 75m for cases with H_t/D=0,0.5,1 respectively). Meanwhile with same \(s_{\text{stiff}}=6\), thicker H_t/D can 6-15
result in huge difference between $s_{\text{stiff}}$ and $k (H_t+H_m)$. The increasing punch-through severity with high top layer thickness is caused by this strengths difference between stiff middle layer and bottom NC clay layer. Consequently, more bearing capacity will lost when the failure plane extends from the stiff layer to the bottom NC layer leading to a quick bearing pressure reduction in Figure 9.

Figure 10 In detail soil strength profiles for cases displayed in Figure 9
When spudcan punches through the stiff layer, no top layer soil can be found underneath which follows the recent experimental tests results (Hossain, 2014), with a stiff soil plug formed under spudcan shown in Figure 11. It can be seen that with thicker clay layer on the top the stiff layer heave is minor compared to double layer case. Also this soft top layer flows back to the cavity much easier and earlier compared to the double layered case ($\alpha = 0$, Group I-3(b) $H_m/D=0.25$). Because of this confinement generated by the top layer, a full punch-through shear failure mechanism occurs and much larger stiff plug can be found for case with $H_t/D=1$ ($\alpha = 0$, Group III-3(b) $H_m/D=0.25$).

When the stiff soil plug has been formed and travel downwards with spudcan it can be seen from Figure 11 that it takes longer for case with $H_t/D=1$ ($\alpha = 0$, Group III-3(b) $H_m/D=0.25$) to diminish the whole plug. Meanwhile the bearing pressure $q_u$ for all the cases gradually approaches the one layer NC clay result and merged together eventually. The second peak can be clearly observed in case with $H_t/D = 1$ due to plug transfers from the bottom of spudcan to the top of it and the sudden change in the shear plane.
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<table>
<thead>
<tr>
<th>Cases in Figure 7</th>
<th>Squeezing mechanism finishing</th>
<th>Punch-through</th>
<th>Plug forming</th>
<th>Plug sliding</th>
<th>Upside plug</th>
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</thead>
<tbody>
<tr>
<td>$H_t/D=0$</td>
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<td>$H_m/D=0.25$</td>
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<tr>
<td>$k=1.5\text{kPa/m}$</td>
<td>$\frac{s_u}{k(H_t+H_m)}=6$</td>
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<td>$H_t/D=1$</td>
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<td>$H_m/D=0.25$</td>
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<tr>
<td>$k=1.5\text{kPa/m}$</td>
<td>$\frac{s_u}{k(H_t+H_m)}=6$</td>
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</table>

Figure 11 Soil failure mechanisms for a smooth spudcan in three-layer clays ($\alpha=0$, Group I-3(b), Group II-3(b) and Group III-3(b) in Table 2)

### 6.6 EFFECT OF THE MIDDLE LAYER THICKNESS RELATIVE TO THE SPUDCAN DIAMETER ($H_m/D$)
In order to demonstrate the effect of the middle layer thickness relative to the spudcan diameter ($H_m/D$) bearing pressure $q_u$ profiles are plotted in Figure 12. For a range of $H_m/D = 0.25, 0.5, 1$ with identical soil properties and top layer thickness respectively of $H_t/D = 0.5, \frac{s_{\text{stiff}}}{k(H_t+H_m)} = 4, k=1$ kPa /m (in Smooth Group II-2(a)Table 2). The corresponding soil flow mechanism throughout the penetration depth is also presented in Figure 13. It should be noted that, thought the soil strength ratio is kept constant for the three cases ($\frac{s_{\text{stiff}}}{k(H_t+H_m)} = 4$), with the increasing $H_m$, the soil strength in the middle stiff layer ($s_{\text{stiff}}$) is increased accordingly.
When spudcan penetrates into the first clay layer, the corresponding rate of increase in bearing pressure, $q_u$, increases as the stiff layer thickness $H_m/D$ increases. For $H_m/D=1$, the curve separates from other curves at a depth 0.2D away from the stiff layer and rises much more sharply compared to other 2 cases owing to the influence of the squeezing mechanism happens in the upper layer. When the foundation approaches the stiff layer, as shown in Figure 13, with $H_m/D$ increasing, a change in soil flow mechanism from a combination of partial squeezing and general shear failure (stiff layer bending) to pure squeezing mechanism can be observed.

Also it can be seen from Figure 12 that the severity of punch-through failure increases as the stiff layer thickness $H_m/D$ increases, leading to a rapid bearing pressure reduction following the peak, which follows the conclusions from previous study on spudcan penetration into stiff clay over soft clay (Hossain and Randolph, 2010, 2010, Gao et al., 2012). For some of the double layered cases with high $H_t/D$ (stiff over soft), the peak can happen much later until the spudcan completely penetrates through the stiff layer due to the reluctance of top stiff layer flowing back into the cavity (Hossain and Randolph, 2010, Gao et al., 2012). However for three layered cases, the peak always happens when spudcan stars penetrating into the stiff layer, owing to the influence of back flow of top soft layer.

With the top soft layer blocked in the cavity, the stiff layer is found heaved up during the punching shear failure. With thicker stiff layer, a higher heave can be observed in Figure 13 resulting less stiff soil left underneath forming the plug. From $H_m/D=0.25$, 0.5,1 after spudcan passes through stiff layer a plug with thickness respectively of $0.9H_m$, $0.8H_m$, $0.5H_m$ is formed and travel downwards with the spudcan.
As spudcan penetration goes further, the bearing pressure increases due to the contribution of bottom NC clay increasing strength for $H_m/D=0.25, \ 0.5$ leading to a relatively short punch-through depth (as marked in Figure 12). On the other hand, the punch-through depth for $H_m/D=1$ is much longer since the bearing pressure only picks up the peak value after the curve merges to one layer NC clay result. For a case like that, too much bearing capacity is lost as soil plug diminishing which couldn’t be compensated by the increasing strength in bottom NC clay.
6.7 EFFECT OF STRENGTH GRADIENT (K)

In order to demonstrate the effect of the strength gradient (k) bearing pressure $q_u$ profiles are plotted in Figure 14. For a range of $k = 1, 1.5$ kPa /m and $\frac{s_{u_{\text{stiff}}}}{k(H_t+H_m)} = 2, 4, 6$

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with identical soil layer geometry respectively of $H_t/D=1$ $H_m/D=0.5$ ($\alpha= 0$, Group III-1(a,b) and Group III-3(a,b) in Table 2). The corresponding soil flow mechanisms throughout the penetration depth are depicted in Figure 15.

![Figure 15](image)

**Figure 14** Effect of strength gradient $k$ on vertical bearing pressure ($\alpha= 0$, $H_t/D=1$, $H_m/D=0.5$, Group III-1(a,b), Group III-2(a,b) and Group III-3(a,b) in Table 2)

It can be seen from Figure 14 that with the same strength ratio $\frac{s_{stiff}}{(k(H_t+H_m))}$ the trend of bearing pressure curves are similar for $k=1$ and 1.5 kPa/m. Also as shown in Figure 15 that with $\frac{s_{stiff}}{k(H_t+H_m)}=6$, the soil flow mechanisms are identical throughout the entire penetration. In order to prove that the soil failure mechanisms and the bearing capacity are independent from soil strength gradient $k$, the bearing capacity factor $N_c$ profiles which reference by using NC clay strength $s_{so}=kxd$ are plotted in Figure 16.

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<table>
<thead>
<tr>
<th>Cases</th>
<th>Squeezing mechanism finishing</th>
<th>Punch-through</th>
<th>Plug forming</th>
<th>Plug sliding</th>
<th>Upside plug</th>
</tr>
</thead>
</table>

Figure 15 Soil failure mechanism for k effect ($\alpha = 0$, Group III-3(a,b) in Table 2)

It can be seen from Figure 16 that all the curves go back to $N_c = 10.5$ which follows the previous results from both numerical study and experiment study (Hossain et al., 2004).

It’s also clear that with $k = 1$ and 1.5 kPa/m the $N_c$ profiles coincide with each other under same strength ratio $\frac{s_{\text{ustiff}}}{k(H_t+H_m)}$ (from 2 to 6), especially during the punching through phase. This indicates that during the penetration, the soil failure mechanisms are similar.

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between cases with different soil strength gradient $k$ and $k$ has no influence on the development of bearing capacity factor $N_c$.

6.8 EFFECT OF THE FOUNDATION ROUGHNESS

The results of all penetration analyses presented so far were obtained considering a fully smooth spudcan base due to the foundation surface is mainly smooth in most of experimental tests from previous study. In order to demonstrate the effect of the foundation roughness bearing pressure $q_u$ profiles are plotted in Figure 17. For a range of $k = 1.5$ kPa /m with identical soil layer geometry respectively of $H/D = 1$ $H_m/D = 0.25$...
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and soil properties \( \frac{s_{\text{stiff}}}{k(H_t+H_m)} = 6 \) (\( \alpha = 0, 1.0, \) Group III-3(b) in Table 2). The corresponding soil flow mechanisms for these two cases at the penetration depth \( d/D = 2.2 \) are also presented in Figure 17.

It can be seen from Figure 17 the roughness of spudcan base has minor effect on the bearing pressure \( q_u \), only showing 4.7% increase on peak bearing pressure and minor difference on the residual bearing pressure. Meanwhile the size and shape of the stiff soil plug underneath are identical for both smooth and rough cases which is indicated by the soil flow mechanism in Figure 17. That also follows the conclusion from previous study on spudcan penetrating into stiff over soft clay profile (Hossain and Randolph, 2010) showing the rough case only gives small extra bearing pressure (<5%). The rough soil-spudcan interface can only affect the very thin layer of stiff plug underneath spudcan which prevents it flowing out from the foundation bottom. However due to the special bottom shape of spudcan, even with a thin layer soil trapped inside (approximately less than 0.2D), the entire soil failure plane remains similar as demonstrated in Figure 18 leading to a similar bearing pressure \( q_u \). Consequently, in all the analyses conducted the effect of spudcan base roughness on its bearing pressure is found to be minimal.
Figure 17 Effect of roughness on vertical bearing pressure ($\alpha = 0, 1.0$ Group III-3(b) in Table 2)

Figure 18 Demonstration of soil flow mechanism at the final stage of penetration

6.9 OVERALL SPUDCAN PENETRATION RESPONSE AND FRAMEWORK
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The combined soil flow effects on the vertical bearing pressure $q_u$ for spudcan penetrating into an NC clay with interbedded stiff layer can be summarized in Figure 19. In order to predict the $q_u$ profile accurately, there are 7 factors identified as key contributors in this study.

(1) Shallow heave or rotational flow: When the spudcan penetration starts, a transition from shallow heave to deep rotational flow around the foundation may occur. Once the deep flow around mechanism was reached, the vertical bearing pressure, $q_u$, profiles coincide with one layer NC clay result, which is the lower bound for the three layered cases studied in this Chapter.

(2) Squeezing mechanism: a squeezing flow of a soft soil layer can be triggered between the underlain stiff layer and the spudcan as the foundation is approaching the stiff layer, leading to a dramatic increase on the vertical bearing pressure $q_u$.

(3) Spudcan punching through the stiff middle layer: When the foundation punching through the stiff layer a direct shear plane on the stiff layer can be identified. A peak bearing pressure may be observed at this stage.

(4) Isolated Plug formation: During the spudcan penetrating into the stiff layer a stiff plug will be formed and with the relatively low strength bottom layer and stiff layer thinning during this process, a reduction in $q_u$ may occur after the peak.

(5) Plug size reduction: The stiff plug underneath the spudcan can diminish with further penetration into the third layer of NC clay, where spudcan bearing pressure converges to the single NC clay result.

(6) Plug moving upwards: With deep penetration of the spudcan with a stiff plug inot the bottom NC clay, the strength of the local NC clay gradually becomes comparable to
the strength of the stiff plug. At this stage, the plug can move upwards and gradually leave the spudcan.

(7) Diminishing plug: The stiff plug diminishes for all the cases analysed once the spudcan penetrates deep in the bottom NC clay, and the residual bearing pressure will coincide with the one layer NC clay result again.

The overall spudcan penetration responses can be summarised in the framework shown in Figure 19 and the corresponding soil flow mechanisms are depicted in Figure 20. There are four different groups (cases a, b, c and d) identified. They are explained in detail below.

From the beginning of penetration, the top layer soil flows back very easily, a full rotation flow is developed and a shallow heave is observed for all the cases. Squeezing mechanism can be identified when spudcan approaches the middle stiff layer. The first material boundary between first layer and second layer deforms differently depending on soil parameters and layer geometry.

With further penetration, the widest brim of the spudcan starts cutting the stiff layer, the thickness of stiff layer under spudcan is reducing, corresponding to the punching through mechanism on bearing pressure $q_u$ prediction in Figure 19. Meanwhile a shear surface bounds a truncated cone of stiff layer material, which is forced into bottom NC clay layer.

After that, the soft soil above the spudcan displaces downwards with the spudcan while the stiff soil trapped under the spudcan forming a plug. The plug acts as one body with the spudcan hence enlarging the foundation size and the shear plane around the spudcan. The bearing pressure, $q_u$, is gradually picked up at this stage. With deeper penetration,
as the local soil strength in the bottom layer becomes higher and becomes comparable to the strength of the soil plug. With further penetration of the spudcan in stronger soil, the plug is gradually squeezed sideways and upwards, and left above the spudcan. Eventually the plug is far enough from the spudcan and the spudcan bearing pressure $q_u$ will join the one layer NC clay $q_u$ profile which is the lower bound of three layered case results.

This Chapter investigated spudcan foundations penetrating into NC clays with an interbedded stiff clay in the middle to identify soil flow mechanisms associated with those factors and to quantify their effects on the bearing pressure $q_u$ profile. It shows the increasing soil strength in bottom NC clay overall contributes to bearing pressure, $q_u$, which cancels the stiff layer thinning effect in a certain degree while the foundation penetrating through the stiff layer (e.g. case a, b, c and d in Figure 19), to prevent the occurrence of punch-through failure. A detailed design frame is established in Chapter 7 to quantify this effect.
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Figure 19 Vertical bearing pressure $q_u$ profile prediction for spudcan penetration on NC clays with a stiff layer in the middle

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<table>
<thead>
<tr>
<th>Examples in Figure 19</th>
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<th>Squeezing mechanism finishing</th>
<th>Punch-through</th>
<th>Plug forming</th>
<th>Plug sliding</th>
<th>Upside plug</th>
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<tbody>
<tr>
<td>Case a example in Figure 19</td>
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</table>

Figure 20 Soil failure mechanism for examples in case a, b, c, d, (Smooth) Group II-3(b) $H_m/D = 1$; (Smooth) Group II-3(b) $H_m/D = 0.25$; (Smooth) Group III-3(b) $H_m/D = 0.25$; (Smooth) Group II-2(a) $H_m/D = 0.25$)

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6.10 REMARKS

This chapter reported results of large deformation finite element analyses investigating all the potential soil failure mechanisms and also punch-through failure during deep penetration of spudcans through NC clays with a strong clay layer sandwiched in the middle. All the analyses were conducted with the spudcan penetrated continuously from the sea bed surface to a depth more than 4D. The resulting soil flow mechanism and bearing pressure profiles were validated against previous centrifuge tests and other relevant published data from numerical analyses. The conclusions can be drawn as listed in Table 3 below.

<table>
<thead>
<tr>
<th>Table 3 Summary of conclusions</th>
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<tr>
<td>$\frac{S_{\text{stiff}}}{k(H_t + H_m)}$</td>
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<td>---------------------------------</td>
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<tr>
<td><strong>(1) Shallow heave or rotational flow</strong></td>
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<tr>
<td><strong>(2) Squeezing mechanism</strong></td>
</tr>
<tr>
<td><strong>(3) Spudcan punching through the stiff middle layer</strong></td>
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</tbody>
</table>

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6.11 REFERENCES


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Lee, K. K. (2009). Investigation of potential spudcan punch-through failure on sand overlying clay soils, University of Western Australia, Perth.


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CHAPTER 7 A DESIGN FRAMEWORK FOR FULL PENETRATION RESISTANCE MODEL OF SPUDCAN IN NC CLAY WITH AN INTERBEDDED STIFF LAYER

7.1 INTRODUCTION

Since 1951 when spudcans were first introduced on the Offshore Company’s Rig 5 (Young et al., 1984), Mobile jack-up rigs have been extensively used for offshore drilling in water depth up to 150m. The potential geohazards during jack-up rigs’ installation processes were discussed by previous studies (Sharples et al., 1989), (Kvitrud et al., 2001), (Jack et al., 2001), (Hunt and Marsh, 2004), (Dier et al., 2004). The unexpected rapid leg penetration has been recognised and studied for decades. However even for now, it’s still a challenge to predict properly the penetration bearing capacity accurately for spudcans, especially on multi-layered soil profiles. This Chapter endeavours to establish a full penetration resistance model for spudcans in NC clays with an interbedded stiff layer, based on the parametric study in Chapter 6.

7.2 CURRENT DESIGN GUIDELINES AND RECENT WORK

7.2.1 SNAME Suggested Design Approach

There were few jack-up failures in the years leading up to 1980 and not much reporting related to ‘punch-through’ failure (Dier et al., 2004). In the late 1980s a Joint Industry 7-1 Centre for Offshore Foundation Systems (COFS)
Project (JIP) was funded to develop a jack-up assessment methodology, and in 1994, SNAME published "T&RB 5-5A Site Specific Assessment of Mobile Jack-Up Units" as a result of that work (Hoyle et al., 2006). Since then, it has been widely used to assess bearing capacities of spudcan foundations which undergo progressive penetration during preloading. In this guideline, owing to the lack of solutions for continuous penetration response, the spudcan ultimate vertical bearing capacity is computed using closed form bearing capacity solutions for various depths of bearing area below sea floor. The bearing capacity formula in SNAME has been empirically derived for surface foundations and does not account for spudcan roughness and shape (SNAME, 2008).

For spudcans on soft over stiff soils, Meyerhof’s solution (Meyerhof and Chaplin, 1953) has been adopted for soil squeezing mechanism of the top soft soil, where the underlying stiff layer is assumed to be infinitely stiff (i.e. rigid).

For spudcans on stiff over soft soils, the evaluation of punch-through potential is computed according to Brown’s solution (Brown and Meyerhof, 1969), where there was no trapped soil plug in the top stiff soil underneath the spudcan considered. However these assumptions do not reflect the reality, though they simplified the analysis.

Moreover, the framework to assess the spudcan bearing capacity profiles in multi-layer soil profiles is basically a combination of the squeezing mechanism with a rigid underlying layer and punch-through criteria without stiff soil plug underneath the spudcan. The framework is based on studies of foundations on two layer soil systems without consideration of soil layer deformation and its effect on foundation capacity.

7.2.2 ISO Suggested Design Approach
In 1997, ISO 19905-1 as an ISO standard for the Site-Specific Assessment of Mobile Offshore Units has been established, using SNAME TR5-5A Revision 2 as the foundation. Since then, this standard has been revised and updated constantly according to the findings from industry case studies and academic research. Therefore the geotechnical design framework is similar in ISO-19905-1 compared to SNAME, only with a certain amount of valuable revisions (Wong et al., 2012). From ISO-19905-1, the shape and depth factors from Hansen’s study (Hansen, 1970) were replaced by those proposed by Skempton (Skempton, 1951) due to a long-established reputation and sufficient accuracy the method can provide. An alteration calculation method, which provides tables of bearing capacity factors (Houlsby and Martin, 2003) for conical foundations with a series of embedment ratios is described in ISO 19905-1. Those factors take direct account of cone shapes, embedment depths, foundation roughness and soil properties. Also more significantly, a rational method to predict the depth at which a flow-round failure mechanism becomes preferential for spudcan penetration (Hossain and Randolph, 2009) is introduced in ISO 19905-1, correcting the reason behind soil backflow from instability of the open hole described in SNAME 5-5A to a preferential local soil flow mechanism. Methods for spudcan penetrating into layered soil profiles are presented in ISO 19905-1 however without much updates from SNAME. Although a series of updated soil failure models are shown in ISO 19905-1 (Figure 1), the appropriate soil model should be used for layered soil to account for effects of punch-through or squeezing. Appropriate equations are not provided in ISO 19905-1. As shown in Figure 1c, underlying stiff layer is still assumed to be infinitely stiff for squeezing as it’s described in SNAME, which is not true in practice. Although
in Figure 1d, 1e and 1f the effects of stiff soil plug are taken into account as a significant update, owing to the lack of detailed formulas and plug diminishing study, more advanced design approach is urged.

(a) Conventional bearing capacity failure: uniform soil

(b) Deep bearing capacity: uniform soil

(c) Squeezing

(d) Punch-through

(e) Punch-through (with trapped soil plug)

(f) Punch-through (with trapped soil plug) and squeezing

Figure 1 Spudcan bearing failure mechanisms (ISO19905-1, 2012)
7.2.3 Other recent work

A series of research studies about the punch-through failure potential has been published in recent years on stiff clay over soft clay (Liu et al., 2005, Hossain and Randolph, 2010, 2010) and sand over clay (Teh et al., 2005, Teh, 2008, Lee, 2009, Teh et al., 2009, Yu et al., 2009). However these methods are not considered as the primary recommendations in the mainstream design guidelines because they are yet to become fully established and need more time to be examined in practice.

Following these conclusions, geotechnical risks for spudcan penetrating into multi-layer system, which is commonly encountered in practice were recently studied (Hossain and Randolph, 2011, Hossain et al., 2011, Hossain, 2014). However, so far there is very limited number of detailed algorithms available for assessing spudcan penetration in a multi-layer soil system. Xie et al. (2010) proposed an algorithm which is similar to SNAME (bottom-up method), assuming each soil layer is independent. This study gathered previous results on one layer and double layer systems, where they choose available methods for each layer; secondly took account of punch-through potential and squeezing potential based on previous studies; then re-plot the penetration curves for multi-layer system. Although it shows acceptable agreement with the field data shown in their paper, no corresponding actual soil flow mechanisms were identified and the prediction accuracy was proved greatly influenced by the method chosen for special soil failure mechanism, which has not fully solved yet.

7.3 NUMERICAL ANALYSIS SETUP AND PARAMETERS
Chapter 7: A Design Framework for Full Penetration Resistance Model of Spudcan in NC Clay with an Interbedded stiff Layer

The schematic diagram of spudcan penetration throughout this study is shown in Figure 2, the soil profile consists of three layers and the soil strength in the 1st and 3rd layer is linearly increasing with depth. In this study all the clay layers were assumed to be undrained (same as the setup in Chapter 6).

![Schematic diagram of spudcan penetration in NC clay with interbedded stiff layer](image)

Figure 2 Schematic diagram of spudcan penetration in NC clay with interbedded stiff layer

The undrained shear strength for NC clay was described as $s_{u1}=s_{u3}=k \times z$ as shown in Figure 2 and $k$ equals to 1kPa/m and 1.5kPa/m respectively, as the typical undrained shear strength gradient $k$ lies within the range 0 to 2 kPa/m in deep water (Lee, 2009). A detailed discussion on numerical analysis input parameters is given in Chapter 6 and same values are used here. In the simulation, $s_{u2}$ was controlled by the strength ratio of stiff middle layer to the surface of bottom layer $\frac{s_{u2}}{k(H_1+H_2)}$ in order to study the punch-through failure. This ratio was varied from 2 to 6. Both soil friction and dilation angles were defined as $0^\circ$ due to the undrained condition and a uniform stiffness ratio $E/s_u = 500$ was selected. The saturated unit weights of clays were both taken as $\gamma_c = 16kN/m^3$.

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and water unit weight was $\gamma_w = 10$ kN/m$^3$. The details of soil parameters are listed in Table 1.

**Table 1 Summary of LDFE analysis performed on three-layer clays**

<table>
<thead>
<tr>
<th>Spudcan base roughness</th>
<th>$\alpha$</th>
<th>0</th>
<th>1</th>
<th>Notes:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normalised top layer thickness</td>
<td>$H_1/D$</td>
<td>0</td>
<td>0.5</td>
<td>1</td>
</tr>
<tr>
<td>Normalised middle layer thickness</td>
<td>$H_2/D$</td>
<td>0.25</td>
<td>0.5</td>
<td>1.0</td>
</tr>
<tr>
<td>Shear strength gradient</td>
<td>$k$ (kPa/m)</td>
<td>1</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>Strength ratio of stiff middle layer and the surface of bottom layer</td>
<td>$s_{u2}$</td>
<td>2</td>
<td>4</td>
<td>6</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>$k(H_1 + H_2)$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2</td>
</tr>
</tbody>
</table>

**7.4 OBJECTIVE OF THIS CHAPER**

Currently a design approach based on soil flow mechanisms is a demand for spudcans penetrating into multi-layered soil deposits, which includes soil layer deformations and continuous soil movement. Based on the soil failure models identified in Chapter 6, a series design formulas will be proposed for spudcans penetrating in NC clay with a stiff clay layer in the middle. The soil flow failure mechanisms corresponding to spudcan penetration depths are proposed in Figure 3. Figure 3a and 3b can be considered as single layer system a shallow with top soil profile and a deep flow mechanism. The failure mechanisms shown in Figure 3c, 3d, 3f and 3d will be studied in detail in this chapter. Full penetration design flow chart will be proposed for practical design.
Chapter 7: A Design Framework for Full Penetration Resistance Model of Spudcan in NC Clay with an Interbedded stiff Layer

Figure 3 Spudcan bearing failure mechanism proposed based on the study in Chapter 6
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7.5 PROPOSED DESIGN PROCEDURE

7.5.1 Step1: Prediction of spudcan bearing resistance profile during its penetration in the first layer

When spudcan is on soft clay over a stronger layer, the soft clay is subject to squeezing mechanism when the spudcan is approaching the stronger layer. The ultimate vertical bearing capacity recommended in almost all the design guidelines or design Algorithms is based on Mayerhof’s study for foundations on an incompressible and cohesive layer (Meyerhof and Chaplin, 1953). For a foundation fully embedded in the top soil layer and sitting at the surface of the soft soil layer (as shown in Figure 4), the recommended formula for the foundation ultimate bearing capacity (SNAME, 2008) is as follows:

\[
F_V = A \left\{ \left( a + \frac{bB}{T} + \frac{1.2D}{B} \right) c_u \right\} + V' \geq A \left\{ N cs c c u \right\} + V' \qquad \text{Equation 1}
\]

where the recommended constants are:

a=5.00
b=0.33

Details for other variables are illustrated in Figure 4.

![Figure 4: Spudcan bearing capacity analysis-squeezing clay (SNAME, 2008)](image)

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Chapter 7: A Design Framework for Full Penetration Resistance Model of Spudcan in NC Clay with an Interbedded stiff Layer

In both the original paper by Meyerhof (1953) and the design guidelines, the strong soil layer is assumed as an infinitely stiff boundary (i.e. rigid boundary) which is not the case in practice. From the recent studies both experimentally and numerically (Hossain et al., 2011, Gao et al., 2012), it has been shown that during the continuous penetration of spudcans, the interface between soft and strong layers will deform to a certain degree before the spudcan reaches the strong layer as shown in Figure 5. Therefore a new soil failure model for squeezing mechanism in a soft soil layer is proposed in Figure 6.

1) Soft-over-stiff soil with strength ratio of 2.4
2) Soft-over-stiff soil with strength ratio of 6.5

Figure 5 Squeezing of soil on soft-over stiff soil in centrifuge tests, vectors from PIV analyses (Hossain et al., 2011)
The depth of deformed soil interface between soft layer and strong layer is defined as $H_{1d}$. Therefore when the foundation penetration depth $d$ reaches $H_1$, the effective thickness of the soft layer material left between the foundation and the strong layer is $(H_{1d}-d)$, which avoids the infinite capacity predicted by Equation 1 at $T = 0$.

The effective thickness of soft material left underneath the foundation is reducing with a certain rate defined as $r$. With further penetration depth $\Delta d$, the soil layer interface depth $H_{1d}$ is changing as $\Delta H_{1d} = (1 - r)\Delta d$, which remarkably related to the strength ratio of two clay layers based on the conclusions in Chapter 6. When this strength ratio $\frac{s_{upper\ layer}}{s_{lower\ layer}}$ is close to 0 (i.e. soft over stiff soil), $r$ should be close to 1. This means the interface doesn’t change and $H_{1d}=H_1$ when strength ratio $\frac{s_{upper\ layer}}{s_{lower\ layer}}$ is close to $\infty$ (i.e. stiff over soft soil), $r$ should be close to 0, which means the material will be always trapped underneath (eg. soil plug). Based on a careful and detailed data fitting analysis.
on the numerical results from Chapter 6, Figure 7 depicts the data fitting curve. A formula for $r$ is obtained as below:

$$r = \frac{\pi/2 - \arctan\left(\left(\frac{S_{\text{upper layer}}}{S_{\text{lower layer}}}\right)^{0.5}\right)}{\pi/2}$$  \hspace{1cm} \text{Equation 2}$$

$$\Delta H_{1d} = (1 - r)\Delta d$$  \hspace{1cm} \text{Equation 3}$$

Following the expression of $(H_{1d} - d)$ in Equation 2, the existing squeezing formula in Equation 1 can be updated as below following the notation in Figure 6:

$$Q_V = A \left\{a + \frac{BD}{2H_{1d} - d} + \frac{1.2d}{D}\right\}_u + V\gamma' \geq A\{N_cS_c \Delta_c u\} + V\gamma'$$  \hspace{1cm} \text{Equation 4}$$

where:

$a=5.00$

$b=0.33$
Chapter 7: A Design Framework for Full Penetration Resistance Model of Spudcan in NC Clay with an Interbedded stiff Layer

In the current design guidelines (Meyerhof and Chaplin, 1953) it has been noted that squeezing occurs when \( D \geq 3.45(H_1 - d)(1 + 1.1 \frac{d}{D}) \) and the upper bound capacity (for \( (H_1 - d) \ll D \)) is determined by the ultimate bearing capacity of the underlying strong soil layer (SNAME, 2008). Figure 8 shows the comparison between Meyerhof’s solution and the currently proposed solution for two cases with both severe soil squeezing (\( H_1/D=0.5, H_2/D=1, k=1.5, \frac{s_u}{k(H_1+H_2)}=6 \)) and less soil squeezing (\( H_1/D=1, H_2/D=0.5, k=1, \frac{s_u}{k(H_1+H_2)}=2 \)). The current proposed formula has shown better predictions on spudcan bearing capacities in both cases.

From Chapter 6, it was observed that the degree of squeezing mechanism in the top soft clay mainly depends on the stiffness of the underneath strong layer relative to the upper layer. With \( H_1/D=0.5 \) and 1, the squeezing mechanism is expected in the zone of 0.5D and 0.76D above stiff layer respectively from Meyerhol’s solution (Equation 1). Figure 8 shows that the currently proposed solution fits well with the LDFE results in both cases, whereas the Meyerhof’s solution gives overestimated bearing pressure without considering the layer interface deformation. Meanwhile Meyerhof solution gives similar degree of squeezing estimations for both cases which is not true.
7.5.2 Step 2: Prediction of spudcan penetration resistance in the second stiff layer

When spudcan penetration depth reaches the initial interface between the first and second layers, there is a thin soft clay layer trapped underneath the spudcan, and the spudcan resistance quickly developed to its peak stage with punch-through failure follows afterwards. The study in Chapter 6 has demonstrated in detail the soil flow mechanisms in this stage, where a direct shear plane in the stiff layer could be identified associated with the top soft soil back-flow above the foundation and bearing contribution from the bottom layer. The contributions to the bearing pressure of spudcan are illustrated in Figure 9, where the peak bearing resistance can occur.
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The gross ultimate vertical bearing capacity of a spudcan foundation at a specific depth can be expressed by:

\[ Q_V = A(q_t + q_b + q_s) \]  

Equation 5

- \( q_t \) is the bearing pressure introduced by the top soft layer back flow

\[ q_t = N_{ct} (k * H_1 + s_{u0}) \]  

Equation 6

\[ N_{ct} = \frac{1}{2} (N_{c, NC clays deep}) = 5.1 \]  

Equation 7

- \( q_b \) is the bearing pressure contributed by the bottom layer.

\[ q_b = N_{cb} s_{ulocal} \]  

Equation 8

The bearing capacity factor that represents soil failure mechanism at the spudcan bottom is a function of soil strength ratio of the stiff soil layer to the soft NC clay in the bottom layer, which was observed in Chapter 6. In order to achieve a good prediction of \( N_{cb} \), soil vertical stress contours extracted from AFENA analysis results are displayed in Figure 10. A vertical shear plan can be observed in stiff middle layer and a bearing type 7-15.
of soil flow mechanism can also be observed underneath stiff soil plug. Further N_{cb} was calculated as per soil vertical bearing pressure with corresponding strength ratio. Cases with different combinations of soil geometries, strength ratios, soil strength gradients are considered and the results are plotted in Figure 11.

Figure 10 Soil vertical stress contour and soil flow for cases with (a) \( \frac{s_{u2}}{k(H_1+H_2)} = 6 \), \( k=1.5 \), \( H_1/D=0.5 \), \( H_2/D=0.5 \) and (b) \( \frac{s_{u2}}{k(H_1+H_2)} = 6 \), \( k=1.5 \), \( H_1/D=1 \), \( H_2/D=0.25 \)

\[
N_{cb} = 5.1 + 0.6 \times \left( \frac{s_{u2}}{s_{ulocal}} - 1 \right)
\]

Figure 11 Bearing capacity factor \( N_{cb} \) \( \frac{s_{u2}}{s_{ulocal}} = 1 - 6 \), \( k=1 - 1.5 \), \( \frac{s_{ulocal}}{y'D} = 0.19 - 0.53 \), \( H_2/D=0.25 - 1 \)

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\[ N_{cb} = 5.1 + 0.6 \times \left( \frac{s_{u2}}{s_{ulocal}} - 1 \right) \quad \text{Equation 9} \]

In Equation 7, \( s_{ulocal} \) represents the soil undrain strength under the plug. It can be expressed as:

\[ s_{ulocal} = s_{u0} + k \times D \times (H_1 + H_{2d}) \quad \text{Equation 10} \]

The depth of deformed soil interface between the middle strong layer and the bottom soft layer is defined as \( H_{2d} \), which is a sum of middle strong layer thickness \( (H_2) \) and plug depth \( (H_p) \) as \( H_{2d} = H_2 + H_p \) as seen in Figure 9.

- \( q_s \) is the shear capacity introduced by the direct shear along the shear plane in the stiff middle layer.

\[ q_s = N_{cs} s_{u2} \quad \text{Equation 11} \]

where \( N_{cs} \) is the ratio of soil shear area along the side and the projective area to the bottom of the spudcan.

\[ N_{cs} = \frac{A_s}{A} \quad \text{Equation 12} \]

\( A_s \) is also a function of \( H_p \):

\[ A_s = (H_1 + H_{2d} - d)D\pi \]

7.5.3 Step 3: Prediction of spudcan penetration resistance in the bottom layer

After the spudcan foundation penetrated through the middle stiff layer, the top soft soil layer above the spudcan and in the middle layer, stiff soil plug beneath the spudcan moves downwards with the spudcan and enter the bottom NC clay layer together. With
further penetration, as the bottom NC clay strength increases with depth, at a certain penetration depth, the local soil strength becomes comparable to the strength of the soil plug (i.e. strength of the middle stiff layer). At this stage, with stronger soil surrounding the plug, the plug is gradually squeezed sideways and eventually leaves the spudcan. This phenomenon is seen as the soil plug diminishing with penetration depth in the bottom NC clay layer till a “clean” spudcan is penetrating in the bottom NC clay. The detailed contributions to the spudcan bearing capacity in the bottom NC clay layer are illustrated in Figure 12.

![Figure 12 Spudcan bearing failure mechanism for Plug minifying](image-url)

\[ q_t = N_{ct} s_{ulocal} \]
\[ q_b = N_{cb} s_{ulocal} \]
Chapter 7: A Design Framework for Full Penetration Resistance Model of Spudcan in NC Clay with an Interbedded stiff Layer

Similar to the framework in the Step 2, as shown in Figure 12, the gross ultimate vertical bearing capacity of a spudcan foundation at a specific depth can be expressed by:

$$Q_V = A(q_t + q_b + q_s)$$

Equation 13

- $q_t$ is the bearing pressure introduced by the top soft layer flow back

$$q_t = N_{ct} (k * H_1 + s_{u0})$$

$$N_{ct} = \frac{1}{2} (N_{c, NC\, clay\, deep}) = 5.1$$

- $q_b$ is the bearing pressure contributed by the bottom layer.

$$q_b = N_{cb} s_{ut, loca}$$

The bearing capacity factor which depicts the soil failure mechanism in the bottom layer is a function of soil strength ratio of the middle stiff layer to the bottom layer, which was demonstrated in Chapter 6. $N_{cb}$ was calculated as per soil vertical stress which is where extracted from AFENA analysis results with corresponding strength ratio and the ratio of middle layer thickness and top two layers’ thickness. The results of case studies in Chapter 6 with different combinations of soil geometries, strength ratios, soil strength gradients are plotted in Figure 13.
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Figure 13 Bearing capacity factor $N_{cb}$ ($s_{u2}/s_{ulocal} = 1~4.5, k=1~1.5, s_{ulocal}/\gamma'D=0.15~0.81, H_2/D=0.25~1$)

$$N_{cb} = 5.1 + 4.9 \times \left( \frac{s_{u2}}{s_{ulocal}} \right) \times \left( \frac{H_2}{H_1 + H_2} \right)$$  \hspace{1cm} \text{Equation 14}

where $s_{ulocal}$ represents the soil undrained shear strength under the plug. It can be expressed as:

$$s_{ulocal} = s_{u0} + k \times D \times (H_1 + H_{2d})$$

The depth of bended soil interface between middle strong layer and bottom soft layer is defined as $H_{2d}$, which is the sum of the middle strong layer thickness and the plug depth ($H_{2d} = H_2 + H_p$).

7.5.4 Soil plug study

When a spudcan penetrates into a new layer of soil, some leftover soil from previous layer will be trapped and dragged into the next layer as the soil flow mechanism observed in Chapter 6. This trapped soil has been recognized as a soil plug and gradually diminishes with further large penetration in the bottom NC clay layer.
During the penetration process in the bottom layer, a good estimate on the size of the plug becomes important since it directly affects the estimation of the spudcan penetration resistance based on the local soil strength $s_{ulocal}$. To provide a continuous penetration response in the bottom clay, the process of the soil plug evolution with spudcan penetration depth needs to be understood.

As discussed in Section 7.5.1, as the spudcan penetrates from the top soft soil to the middle stiff soil, the thickness of the soft material left underneath the spudcan is reducing with a certain reduction rate defined as $r$ corresponding to the incremental penetration depth $\Delta d$. Following the same logic of the soil layer interface $LI-1$ depth $H_{1d}$ is changing with penetration depth $d$ as $\Delta H_{1d}=(1-r)\Delta d$, the change of soil interface $LI-2$ can be estimated as below, where $r$ can be estimated using Equation 2.

$$\Delta H_{2d}=(1-r)\Delta d$$  

Equation 15

The soil plug $H_p$ has been defined as the soil from the previous layer trapped under the spudcan after entering the current layer. It was assumed that once the spudcan entered the third layer the first soil layer would not be considered as a soil plug, since the soil from the first soft layer has disappeared underneath the spudcan, which was observed in the cases studied here (see the soil flow mechanisms displayed in Chapter 6).

The studies on the soil plug formation and the soil layer interface deformation are depicted in Figure 14 and Figure 15 as two examples during the entire penetration process. Curve C1 shows the thickness of the first soil layer underneath the spudcan. It disappeared nearly entirely when spudcan enters the second stiff layer, though a thin soft material may still exist after the spudcan is in the stiff layer. Curve C2 displays the thickness of the second soil layer underneath the spudcan. It starts to get thinner and
thinner when the spudcan travels down to the second layer and eventually it disappears when spudcan penetrating into the third layer deeply. Curve C3 shows the depth of the first soil layer interface (the bottom of first layer). It initially positioned at $H_1/D$ and being pushed downwards by spudcan penetration. The same for Curve C4 which represents the depth of the second soil layer interface (the bottom of second layer).

The results of soil plug and soil layer interface deformation observed and measured from the LDFE/RITSS studies are compared with the estimations using Equations 2, 3 and 4. It can be seen that the estimations match very well with the LDFE/RITSS results, except when the spudcan penetrated deeply into the third layer and the stiff soil plug is almost eliminated. That is due to the special geometry at the bottom of the spudcan. A small amount of stiff soil always trapped around the tip of the spudcan. However as per the conclusions from chapter 6, when the soil plug thickness is less than 0.2D the entire soil failure plane remains similar as the spudcan penetrating into NC clay, leading to a similar bearing pressure. Therefore it can be concluded the proposed soil plug estimation method is satisfactory.
Chapter 7: A Design Framework for Full Penetration Resistance Model of Spudcan in NC Clay with an Interbedded stiff Layer

Figure 14 Soil plug study \( k=1.5, \frac{s_u}{k(H_1+H_2)}=6, H_1/D=0.5, H_2/D=0.5, s_u\) upper layer/su lower layer =0~11.7)
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7.5.5 Summary: The output of the proposed prediction method of spudcan bearing resistance

Based on the proposed estimation methods of soil layer interface deformation/soil plug evolution stated above, the continuous spudcan penetration resistance during the entire penetration process can be predicted. A flow chart to predict the spudcan bearing resistance is summarised in Figure 16.
Chapter 7: A Design Framework for Full Penetration Resistance Model of Spudcan in NC Clay with an Interbedded stiff Layer

**Input:** D, $s_{u0}$, $k$, $\frac{s_{u2}}{k(H_1+H_2)}$, $H_1/D$, $H_2/D$, $\Delta d$, total steps

**Spudcan bearing resistance prediction within first layer**

**Squeezing mechanism:**

- $s_{uper\ layer} = \text{penetration depth} \times k + s_{u0}$
- $s_{lower\ layer} = s_{u2}$
- $r$ (Eq. 2)
- $H_1d/D$ (Eq. 3)
- $Q_V$ (Eq. 4)

**Spudcan bearing resistance prediction within second layer**

**Punch-through mechanism:**

- $s_{uper\ layer} = s_{u2}$
- $s_{lower\ layer} = (H_1 + H_2d) \times k + s_{u0}$
- $R$ (Eq. 2)
- $H_2d/D$ (Eq. 15)
- $Q_V$ (Eq. 5)

**Spudcan bearing resistance prediction within third layer**

**Soil plug eliminating mechanism:**

- $s_{uper\ layer} = s_{u2}$
- $s_{lower\ layer} = (H_1 + H_2d) \times k + s_{u0}$
- $r$ (Eq. 2)
- $H_2d/D$ (Eq. 15)
- $Q_V$ (Eq. 13)

**Draw a constant line for the late third layer based on previous study for NC clay.**

Hu et al. 2002

**Averaging the assembled results and determine the whole curve.**

---

**Figure 16** Summary of the proposed prediction method of spudcan bearing resistance

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7.6 MODEL VALIDATION

The proposed bearing capacity prediction method has been validated by a large number of numerical data with a wide range of parameters (k=1~1.5, H1/D H2/D=0.25~1, \( \frac{s_{u2}}{k(H_1+H_2)}=2~6 \)). As per the demonstration in Figure 16, the whole process heavily relies on the material interface deformation/ residual plug thickness estimation, which is a stepwise calculation. Therefore a simple numerical program based on eg MATLAB etc. is recommended to conducted the calculation. The incremental penetration depth \( \Delta d \) was selected by being refined until the results converges. An example is presented in this section to explain in detail about the proposed method as below:

Input: \( k=1.5, \)

\[ \text{D}=12\text{m} \]

\[ \text{H}_1/\text{D}=0.5 \]

\[ \text{H}_2/\text{D}=0.5, \]

\[ \frac{s_{u2}}{k(H_1+H_2)}=2, 4, 6 \]

Output:

- Reduction rate of the material thickness left under spudcan \( r \) has been calculated as per equation 2 along the penetration depth with the soil strength ratio dynamically updated iteratively each step, the results plotted as below in Figure 17.
Chapter 7: A Design Framework for Full Penetration Resistance Model of Spudcan in NC Clay with an Interbedded stiff Layer

Figure 7.27: Reduction rate of the material thickness left under spudcan $r$ results ($k=1.5$, $\frac{s_{u2}}{k(H_1+H_2)}=2$ to 6, $H_1/D=0.5$, $H_2/D=0.5$)

- Soil plug therefore can be determined and plotted in Figure 7.28:

Figure 7.28: Soil plug from stiff layer thickness estimation results ($k=1.5$, $\frac{s_{u2}}{k(H_1+H_2)}=2$ to 6, $H_1/D=0.5$, $H_2/D=0.5$)

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Finally the bearing capacity curves for these three cases can be estimated along three different layers based on the estimation method above and are displayed in Figure 19. The LDFE/RITSS results for these three cases have also been plotted and compared with the estimated results. A good match can be found in Figure 19.

![Figure 19](image)

Figure 19 Comparison of load-penetration response between current FE analysis and estimated results $k=1.5$, $\frac{s_{uz}}{k(H_1+H_2)}=2-6$, $H_1/D=0.5$, $H_2/D=0.5$

Validation also conducted with previous centrifuge test (Hossain et al., 2011) together with current numerical analysis.

Input: $k=0.6$, 7-28

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\[ D = 12 \text{m} \]

\[ \frac{H_1}{D} = 0.375 \]

\[ \frac{H_2}{D} = 0.575 \]

\[ \frac{s_{u2}}{k(H_1 + H_2)} = 3.8 \]

It can be seen that the estimated result shows almost the same vertical bearing pressure along the penetration and similar peak position as the centrifuge test result.
Figure 20 Comparison of load-penetration response between proposed method current FE analysis and previous centrifuge study (Stiff clay layered sandwiched between soft NC clay with soil strength details as per Figure 8 in (Hossain et al., 2011). D=12m)

7.7 REFERENCES


Chapter 7: A Design Framework for Full Penetration Resistance Model of Spudcan in NC Clay

with an Interbedded stiff Layer


Lee, K. K. (2009). Investigation of potential spudcan punch-through failure on sand overlying clay soils, University of Western Australia Perth.


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8.1 INTRODUCTION

In this thesis, large deformation finite element (LDFE) analyses have been carried out to study the bearing capacity of spudcans, on multi-layered soil profiles. Three configurations of three-layer soils have been investigated, including i) a uniform stiff clay with an interbedded uniform soft clay layer (i.e. stiff-soft-stiff clay), ii) a uniform soft clay with an interbedded uniform stiff clay layer (i.e. soft-stiff-soft clay); iii) an NC clay with an interbedded uniform stiff clay layer.

A wide range of soil properties were selected based on the survey data of offshore deposits. The effects of multiple soil parameters, such as soil layer strength ratio and soil layer thickness ratio, on the spudcan penetration responses were studied, and the soil failure mechanisms were explored.

Numerical results were calibrated by existing centrifuge data, and then soil flow mechanisms throughout the entire penetration process were presented and studied. Bearing capacity curves were plotted and a new design approach to predict bearing responses were proposed which is based on the extensive parametric study in this research. The LDFE/RITSS analyses have provided the link between the spudcan bearing response and the soil flow mechanisms during the continuous penetration of spudcan. Thus the predictive framework is mechanism based, which includes soil layer deformations and continuous soil movements. Based on the soil failure models identified in the study, a series design formulas are proposed corresponding to the
spudcan penetration depths. The proposed framework is tested with a large database of model test results. Therefore it can be used as a design tool for assessing “punch-through failure” risk when jack-ups with spudcan foundations are installed on the seabed with multilayered clays.

8.2 CONCLUSIONS-MAIN FINDINGS

8.2.1 A uniform stiff clay with an interbedded uniform soft clay layer

Chapter 4 presents the explorations for spudcan foundations in a seabed with a soft middle layer embedded into uniform clays on spudcan penetration to identify the soil flow mechanisms and the sensibility of each soil parameters.

It was found that with increasing the middle soft layer thickness \( H_m \), the peak bearing capacity decreased. With thinner top layer, increasing middle soft layer will lead to higher punch-through potential. When \( H_t/D=1 \), \( H_m/D=0.25 \) no punch-through was observed. However with thick top layer (\( H_t/D=1 \)) even a thin middle soft layer will trigger punch-through failure. Increasing soil strength ratio leads to a lower punch-through risk. Punch-through failures are triggered with strength ratio \( S_{un}/S_{ut} = S_{un}/S_{ub} = 0.25 \), 0.375 0.5 while \( S_{un}/S_{ut} = S_{un}/S_{ub} \) reached 0.75 no obvious peak bearing capacity occurs.

The overall bearing capacity is increasing by increased soil strength ratio leading to different value for peak and bottom bearing capacity and also got slightly influence on the depth of peak bearing capacity. However compared to the total depth of punch-through, this effect is negligible. The normalised shear strength, \( s_u/\gamma'D \) plays a significant role on the transition depth for soil back-flow into cavity. A design chart for punch-through depth for stiff-soft-stiff soil profile was proposed together with corrected formulations to estimate soil flow cavity depth during spudcan penetration.
8.2.2 A uniform soft clay with an interbedded uniform stiff clay layer

Chapter 5 presents the explorations for spudcan foundations in with a strong middle layer embedded into uniform clays. Bearing factor Nc profile prediction for this soil configuration has been summarized with detailed soil flow mechanisms being identified as below:

(1) When the penetration starts, a transition from shallow heave to deep flow around foundation may occur where the Nc profile starts to climb up from N_{surface}.

(2) Afterwards a squeezing flow will be triggered between the stiff layer and spudcan, leading a dramatic increase on Nc.

(3) When the foundation punching through the stiff layer a direct shear plain on the stiff layer will be identified.

(4)/(5) The Nc increase due to the contribution of stiff soil layer while the stiff layer thickness is decreasing resulting in a reduction on Nc. Different effects combination will cause different Nc profile path.

(6) When the soil starts flow back into the cavity after punching though the stiff layer and losing overburden effect, a reduction on Nc can be observed.

(7) The stiff plug underneath the spudcan will be minified with further penetration.

(8) Part of the plug will diminish.

(9) Part of the plug will be stabilized underneath the spudcan and act as part of the foundation.

Each of the soil flow mechanism was investigated associated with soil parameters which discussed in chapter 4. The effect of those parameters on the Nc profile were 8-3
quantified. Finally corrected formulation to estimate the peak capacity values and location when they occur are provided at the end of the chapter.

8.2.3 An NC clay with an interbedded uniform stiff clay

Chapter 6 presents the explorations for spudcan foundations in with a strong middle layer embedded into NC clays on spudcan penetration. The resulting soil flow mechanism and bearing pressure profiles were validated against previous centrifuge tests and other relevant published data from numerical analyses. Then to carry on the work, which have been discussed in chapter 5, extensive LDFE cases were conducted to understand the soil failure mechanism and bearing capacity when spudcan penetrating into this soil configuration, which is prevalent in ocean floors around the world. The effect of the major factors (including soil strength ratio, normalised top layer thickness, normalised middle layer thickness, soil strength gradient and the roughness of spudcan base) are studied and quantified for each soil flow mechanism throughout the entire spudcan penetration process. The conclusion obtained in Chapter 6 leads to a pathway to Chapter 7, which proposed estimation methods of soil layer interface deformation/soil plug evolution with proposed formulations to obtain the entire spudcan penetration profiles, showing encouraging agreement between the predicted bearing capacities and LDFE analyses results.

8.3 FURTHER RESEARCH

8.3.1 LDFE Analysis Database

In this research, the input parameters adopted in the LDFE analysis were selected to cover a broad range hence the new design approach based on the numerical results of this study could be applicable to majority of the cases, where the potential punch-
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through failure exists. However there are still limitations/borders for all the parameters. In real world, the combination of soil parameters and layer geometry for multi-layer soil is much more diversified. Therefore more LDFE analysis may be required to cover a broader range of parameters (e.g. soil strength ratios, spudcan diameters and soil layer geometry etc.). As such a database contains all the LDFE analysis results which links back to the proposed formulation could be valuable to establish. As more data/LDFE analysis being conducted and imported into the database, the proposed formulations will stay dynamic and be adjusted/updated to keep the accuracy and applicability to practical cases.

8.3.2 Calibration with industrial/in-situ data

The LDFE analysis and resulted formulation are based on input soil data which have been pre-defined. However in real world soil data need to be obtained through in-situ sampling and lab testing. The proposed formulations will need to be calibrated with site data and practices of obtaining and interpreting soil data for spudcan installation. Joint Industry Project may be proposed in the future to obtain industrial filed data/full-scale test data.

8.3.3 Advanced Soil Models

In this study, the clay was assumed to be linear elastic-perfectly plastic (cohesive) material with Tresca failure criterion and sand layer was treated as a linear elastic-perfectly plastic (frictional) material with Mohr-Coulomb failure criterion. The proposed formulations are subject to clay only. However calcareous sand and very loose silica sand are also prevalent especially in Australia offshore field. The advanced soil
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models capable of conducting LDPE analysis for different type of soils (especially to simulate the compressibility and crushability of sand) are urged.