ROTATIONAL STIFFNESS AND BEARING CAPACITY VARIATION OF SPUDCAN UNDER UNDRAINED AND PARTIALLY DRAINED CONDITION IN CLAY

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Summary

Jack-up platforms are widely used to explore oil and gas resources offshore. During the operation of a jack-up, the interaction between the soil and spudcan foundation would greatly affect the distribution of bending moment on the legs, the operation and the assessment of stability of the jack-up. Literature review reveals that the strain-hardening force-resultant model developed by Houlsby and Martin (1994) is an effective model to examine spudcan-soil interaction. This model assumed undrained condition for clay, but how the soil responds under partially drained condition when the jack-up is standing at a certain place for a period of time needs to be evaluated.

The first step of the present study is to investigate the rotational stiffness variation under undrained and partially drained condition using centrifuge modeling technique. To assess the initial stiffness, the results of six tests were compared with existing elastic stiffness theories. A relationship which is based on the fitted curve with test data and representing the rotational stiffness variation with time was presented. Thus, the rotational stiffness variation can be embodied in the force-resultant model with this generalized relationship when the soil around the spudcan experiences a period of consolidation.

The yield surface of the Houlsby and Martin (1994) ’s model was verified with centrifuge scale models since the previous studies were done using small scale models under 1g condition. Loading and unloading tests on spudcan were
conducted in the centrifuge to confirm the low unloading-reloading gradient ratio which is an important component of the similarities between the force-resultant model and the modified Cam Clay model derived by Martin (1994) and Tan (1990). The results from eleven centrifuge tests under undrained conditions were plotted in the normalized yield space. It is found that the data fit well with the yield surface. Further centrifuge tests were done to investigate the effects of soil consolidation when the jack-up is operating for a few years after the initial installation of the spudcan. It is found that these yield points will lie outside the yield surface if the initial bearing capacity, \( V_{Lo} \), is used in the force-resultant model after a period of consolidation.

As the yield surface is controlled by the bearing capacity at the designated depth, the results from existing bearing capacity theories were compared with the test data under undrained condition. It is found that the approach developed by Houlsby and Martin (2003) is more accurate than the other methods. This method will provide a basis for the later study of bearing capacity with time effects.

In dealing with time effects, the problem will be how to embody the time effects in the force-resultant model so that the yield points under partially drained conditions can still lie on the yield surface. In clay with strength linearly increasing with depth, the bearing capacity variation under partially drained condition is generalized as a hyperbolic function with time. With this empirical function, the yield points lying outside the yield surface due to consolidation can be mapped into the yield surface.
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Notation

Chapter 1

E elastic modulus of the jack-up legs
f₀ natural frequency of the platform with fixed footings
f₁ natural frequency of the platform in the field
fₚ natural frequency of the platform with pinned footings
I second moment of area of Jack-up legs
Kθ rotational stiffness provided by soil on the spudcan
L length of Jack-up legs

Chapter 2

A maximum area of the spudcan
A' effective footing area
Aₑ laterally projected embedded area of spudcan
aₕ association factor of horizontal force
aₘ association factor of moment force
b width of strip footings
B effective spudcan diameter
B' effective footing width
bₑ effective footing width
c cohesion of clay
Cu undrained shear strength of the soil
c₁₀ undrained shear strength at spudcan tip
Cum undrained shear strength of clay at mudline level
cₒ₀ undrained shear strength of the soil at maximum bearing area of spudcan
d depth of the soil
D  diameter of the circle
D_r  relative density of sand
e  eccentricity of loading
E'  effective elastic modulus of soil
e_o  eccentricity parallel to width side of the footing
eQ  moment caused by the eccentricity of vertical forces
E_u  undrained elastic modulus of clay
f_p  dimensionless constant describing the limiting magnitude of vertical load
f_r  reduction factor of rotational stiffness
F_{vh}  vertical leg reaction during preloading
G  shear modulus of the soil
G_h  horizontal shear modulus of sand
G_r  rotational shear modulus of sand
G_v  vertical shear modulus of sand
H  horizontal forces applied on the footing
h  embedment of spudcan(from mud line to the maximum area of spudcan)
h_o  factor determining the horizontal dimension of yield surface
I_r  rigidity index
k_b  bottom spring stiffness of the footings
K_h  horizontal stiffness of the footing
K  rotational stiffness
K_{rec}  rotational stiffness of rectangular footing
k_o  single side spring stiffness of the footing
K_v  vertical stiffness of the footing
K_v*  modified vertical stiffness of the spudcan
l  half dimension of rectangular footing
L'  effective footing length
M  moment applied on the footings
M  moment applied on the footing
m_o  factor determining the moment dimension of yield surface
M_o  ultimate overturning moment
p_a  atmospheric pressure
p_o'  effective overburden pressure at the maximum area of spudcan
Q  point load
Q_{e}  moment per unit length
Q_{vh}  vertical forces applied on the leeward spudcan
R  footing radius
r  uplift ratio
r_f  failure ratio
S_ri  initial rotational stiffness
s_u  undrained shear strength of the soil
S_{u,ave}  average undrained shear strength of clay
u  vertical displacement of the footing
\( v \) soil Poisson's ratio
\( V \) vertical forces applied on the footing
\( \nu' \) drained soil Poisson's ratio
\( V_{uo} \) ultimate bearing capacity of the footing
\( V_{om} \) peak value of \( V_{uo} \)
\( \nu_u \) undrained soil Poisson's ratio
\( w \) horizontal displacement of the footing
\( w_{pm} \) peak plastic vertical penetration
\( y \) distance of loading point to the center line of longer sides
\( \alpha \) rotation angle of the footings
\( \alpha_1 \) roughness factor
\( \alpha_i \) initial rotational angle
\( \alpha_i' \) modified rotational angle after taking account of embedment effects
\( \beta \) ground inclination angle (in radian)
\( \beta_1, \beta_2 \) round off factor of the yield surface
\( \beta_3 \) curvature factor of plastic potential surface at low stress
\( \beta_4 \) curvature factor of plastic potential surface at high stress
\( \beta_c \) equivalent cone angle of spudcan
\( \delta_p \) friction angle between soil and structure
\( \eta = y/b \)
\( \theta \) rotational displacement of the footing
\( \theta_1 \) angle between longer axis of footing and horizontal component of loading
\( \xi \) ratio of length and width
\( \rho \) gradient of shear strength increase of clay
\( \Omega \) reduction factor
\( \phi \) friction angle of the soil
\( \phi_{vh} \) resistance factor for foundation capacity during preload

Chapter 4

\( G \) shear modulus of soil
\( C_u \) undrained shear strength of clay
\( k_a \) at rest earth pressure factor
\( k_{vo} \) initial rotational stiffness of clay at \( t=0 \)
\( k_{rt} \) rotational stiffness of clay at time \( t \)
\( M_{ini} \) initialized bending moment on spudcan
\( n \) unloading ratio
\( OCR \) over consolidation ratio
\( R \) radius of spudcan
\( t \) consolidation time
\( \theta_{ini} \) initialized rotational angle of spudcan
\( \nu_0 \) at rest soil Poisson’s ratio
\( \beta \) rotational stiffness multiplier
\( \phi' \) effective friction angle of soil

Chapter 5
B    diameter spudcan or width of trench
C_{us} undrained shear strength of clay at spudcan penetrated depth
D    penetration depth of spudcan or depth of trench
F_{ur} gradient of unloading-reloading line
F_{vir} gradient of virgin penetration line
k    curvature of a curve
N    stability number
γ'   submerged unit weight of soil
κ    gradient of swelling and recompression line
λ    gradient of normal and critical state lines

Chapter 6

V_0    bearing capacity of spudcan immediately after penetration
V_t    bearing capacity of spudcan at time t after penetration
ξ    bearing capacity variation multiplier
1 Introduction

1.1 Study background

1.1.1 Jack-up platform and spudcan

Since the first jack-up built in the 1950’s, jack-up platforms have been used intensively all over the world. They are generally used for exploration, accommodation, assisted drilling, production and work/maintenance in offshore oil fields. Jack-up platform is a movable offshore structure which is towed to the site, after which the legs are lowered and the spudcans penetrated into the seabed. One full view of a jack-up platform is shown in Fig. 1-1. The foundation of a jack-up rig consists of spudcans which can be in different shapes (see Fig. 1-2.). To keep the jack-up platform stable, a process called preloading is utilized to penetrate the spudcan into the seabed. After the spudcan is installed to a certain depth during preloading, water will be pumped out of the hull resulting in unloading of the rig. During operation, the jackup will work under self-weight and environment loads. The installation process is presented in Fig. 1-3.

1.1.2 Definition of spudcan fixity

Spudcan fixity is the restraint provided by the soil to the jack-up spudcan. It is often represented by vertical, horizontal and rotational stiffness of the soil. It is an important consideration in jack-up unit assessment. As has been known, the field
conditions cannot be completely included during the design stage of the units and geotechnical properties of the seabed varies from place to place. Thus, the assumed soil parameters may not represent the actual condition in the field and the jack-up rig needs to be specifically assessed according to the site investigation or past data obtained from the surrounding areas. Generally four methods are used to simulate the soil stiffness around the footing, that is, pinned, encastred, linear spring and plasticity model. The rotational fixity often dominates the jack up behavior under combined loading; rotational stiffness is generally regarded as the most important factor influencing the spudcan fixity. Since 1980’s, spudcan fixity has been considered as a significant topic for further studies in practice and research. A few improvements were made in the last twenty years. The static and dynamic fixity are mainly defined as follows.

Static fixity is defined as the ratio of rotational stiffness of spudcan to the rotational stiffness considering both spudcan and leg-hull connection, expressed as follow:

\[
\frac{K_\theta}{K_\theta + \frac{EI}{L}} \quad (1.1)
\]

where \(K_\theta\) is rotational stiffness provided by soil on the spudcan on the seabed, \(E\), \(I\), \(L\) are the elastic modulus, second moment of area and length of the leg, respectively.

Dynamic fixity is defined as the ratio of natural frequencies and expressed as follow:

\[
\frac{f_{n}^2 - f_0^2}{f_f^2 - f_0^2} \quad (1.2)
\]
where \( f_n, f_0, f_f \) are natural frequency of the platform considering the field status, pinned and fixed condition, respectively.

### 1.1.3 Why study fixity?

When a jack up rig is designed and fabricated, engineers do not know the exact sea and seabed information. They often assume some values used in some particular areas or accept the data provided by the client. If the exploration work goes to another location, the environmental load changes and the soil properties of the seabed vary. Hence the previous assumption may not hold. These rigs should therefore be assessed again with appropriate site-specific soil parameters.

Consideration of spudcan fixity during site assessment can improve the performance of a jack up unit. Statically the moment-resistance capacity of the spudcan due to fixity can lead to redistribution of bending moment so that the moment at the leg-hull connection would be reduced. Meanwhile, fixity also reduces the horizontal displacements of the unit, as reported by Santa Maria (1988). The static effects can be illustrated by Fig. 1-4.

One of the well-known examples was the modification of MSC CJ62 design (Baerheim 1993). Due to unfavorable soil conditions, the original design needed to be revised to fulfill the field requirements in the Norwegian sector of the North Sea. Statoil together with Sleipner Vest Development analyzed the jack-up rig and decided to equip the spudcans with skirts. The modification showed significant improvement in the performance of this rig, benefiting from the improved fixity.
The analysis found that the improved fixity reduced the stresses in the leg-hull connection. It is thus beneficial to further study the issue of spudcan fixity.

1.2 Objectives and scope of study

Beyond the conventional pinned, encasted and linear spring assumption, the work hardening plasticity model has been proven to be the most comprehensive model to be incorporated into the structural analysis of jack-up rigs to date. The elastic stiffness resulting from Bell (1991)’s numerical study is used in this model. However, the elastic rotational stiffnesses from conventional theory and Bell’s study have not been assessed previously. Moreover, the consolidation effect has not been considered in these rotational stiffness. The yield surface of this model was developed with experimental results of small scale spudcan under 1g condition, whether it is applicable to large scale spudcan or not is not clear. The yield surface is governed by the ultimate bearing capacity of the spudcan. There are many existing bearing capacity theories. Little work has been done on the application of these theories when the jack-up experiences relatively long operation time at one place. Thus, the bearing capacity variation with consolidation time and its effect on the yield surface of the force resultant model needs to be investigated.

The objectives of this study will be to assess the existing rotational stiffness and bearing capacity theories, verify the yield surface of the strain-hardening force resultant model of realistic prototype scale of spudcan under undrained condition
in the centrifuge and better understand the spudcan fixity under partially drained condition. Finally an effective way will be provided to analyze the rotational stiffness variation and bearing capacity variation of spudcan under partially drained condition. Thus, the scope of the work carried out is as follows:

1) Assessment of rotational stiffness of spudcan in kaolin clay in centrifuge tests with conventional method and Bell’s FEM results.

2) Physical study of rotational stiffness variation of spudcan in clay in the centrifuge under partially drained condition.

3) Verification of yield surface of strain-hardening force-resultant model of realistic prototype scale of spudcan in clay under undrained condition in the centrifuge.

4) Comparison of different approaches to obtain the ultimate bearing capacity of spudcan under undrained condition.

5) Experimental study of bearing capacity variation of normally consolidated clay in centrifuge under partially drained condition. An empirical approach is developed to incorporate the time effects into the existing force-resultant model so that the current model can still hold without variation of its main components.
Fig. 1-1: Plan and elevation view of jack-up platform, Majellan (Courtesy of Global Santa Fe)
Fig. 1-2: Types of spudcans developed (CLAROM 1993)
Fig. 1-3: Jack-up installation progress (Young 1984)

Fig. 1-4: The effects of spudcan fixity (Santa Maria 1988)
2 Literature review

2.1 Introduction

As discussed in Chapter 1, the main objectives of this study is to assess the existing rotational stiffness theories and ultimate bearing capacity theories, verify the yield surface of force-resultant model and derive the rotational stiffness and bearing capacity variation with time as the soil consolidates. There are three main sections in this chapter; namely, foundation stiffness study, yield behavior and ultimate bearing capacity.

In the foundation stiffness study, the work on offshore and onshore footing stiffness are reviewed and generalized. Some of the numerical verification work on stiffness is included in the review.

The review of yield behavior studies are classified as experimental and numerical. As some of the numerical models are derived based on experimental data, these theories are shown in both parts. Three main models are introduced in this section: the SNAME (2002) recommended model, Langen and Hooper(1993)’s model and Houlsby&Martin(1994)’s model.

Several theories on ultimate bearing capacity will be reviewed in the third part. These theories will provide the basis for the subsequent bearing capacity study. Jack up rigs are currently assessed based on the “recommended practice for site
specific assessment of mobile jack-up units” issued by the Society of Naval Architecture and Marine Engineer (SNAME 2002). The assessment is done under three categories, that is, preload, bearing capacity, and displacement check (see Fig. 2-1) (Langen 1993). The fixity is included in the latter two steps.

Preloading check is often based on the assumption of ultimate bearing capacity of soil under extreme conditions. In subsequent check, the soil-structure interaction is generally simulated as pinned. Single degree of freedom, multi-degree of freedom methods or random analysis would be engaged to obtain the jack-up response. Sliding may occur in the windward legs, and this needs to be checked.

The contents related to rotational stiffness, yield surface and bearing capacity of spudcan in SNAME (2002) will be reviewed.

The second guideline, API RP2A-WSD (2002), mainly caters for gravity or mat footings. In this guideline, the classical elastic soil stiffnesses as summarized by Poulos & Davis (1974) are recommended, but it does not account for the large deformation under combined loads. Bearing capacity calculation of shallow foundation follows the procedures by Vesic (Winterkorn 1975), and takes into account the foundation shape, load inclination, embedment depth, base inclination and ground inclination effects. The relevant materials will be reviewed in the appropriate sections.
2.2 Foundation stiffness study

2.2.1 Conventional foundation stiffness study

Several stiffness studies for onshore footings are briefly and chronologically reviewed in this section.

Borowicka (1943) derived the earliest equations for rigid footings on an elastic half space. For rigid circular footings, the moment rotation is expressed as:

\[ \tan \alpha = \frac{3}{4} \frac{1 - \nu^2}{E} \frac{M}{R^3} \]  
(2.1)

For strip footing, \( \tan \alpha = \frac{8}{\pi} \frac{1 - \nu^2}{E} \frac{M}{b} \)  
(2.2)

where \( \alpha \) is the rotational angle; \( M \) is the moment applied on the footing; \( R \) is the footing radius, \( b \) is the strip width.

Tettinek-Matl (1953) published the rotational response of flexible footing on an elastic half space. For rectangular flexible footings,

\[ \tan \alpha = \frac{3}{\pi} k(\zeta, \eta) \frac{1 - \nu^2}{E} \frac{Q_e}{b^2} \]  
(2.3)

where \( Q_e \) moment per unit length, \( \zeta = l/b \), the ratio of length and width, \( \eta = y/b \), \( y \) is the distance of loading point to the center line of longer sides. Two special cases were given.

For the case \( \eta = 1 \) (edge),
\[ k(\varsigma, \eta) = k_1(\varsigma) = \varsigma \left( \ln \frac{1 + \sqrt{1 + \varsigma^2}}{\varsigma} + \varsigma - \sqrt{1 + \varsigma^2} \right) \]  

(2.4)

For the case \( \eta = 0 \) (middle section),

\[ k(\varsigma, \eta) = k_0(\varsigma) = \varsigma \left( \ln \frac{2 + \sqrt{4 + \varsigma^2}}{\varsigma} + \frac{1}{2} \left( \varsigma - 2 \sqrt{4 + \varsigma^2} \right) \right) \]  

(2.5)

For circular flexible footings,

\[ \tan \alpha = \frac{16}{3\pi^2} \frac{1 - \nu^2}{E} \frac{Q_e}{R^3} \]  

(2.6)

Majer (1958) proceeded with the predecessor’s work and summarized his main finding into a rotational stiffness chart for rectangular footings on an elastic half space, \( k_{rec} \), as

\[ \tan \alpha = k_{rec} \frac{1 - \nu^2}{E} \frac{Q e_b}{lb^3} \]  

(2.7)

where \( l, b \) are half dimensions of footing, \( e_b \) is eccentricity parallel to side \( b \), \( Q \) point load. The values for \( k_{rec} \) are given in Fig. 2-2.

If the effects of embedment and side friction of the footing was included, the initial stiffness would be modified. The modified initial rotational angle can be represented as follow:

\[ \frac{\alpha'_{i}}{\alpha_{i}} = 1 - \Omega = 1 - \frac{E_b h_p}{M} = \frac{M}{M} \]  

(2.8)

where \( \alpha'_{i} \) is the modified rotational angle after taking into account of embedment effects, \( \alpha_{i} \) is initial rotational angle; \( \Omega = 1 + \frac{L}{2h_p} \left( \tan \delta_p + \frac{T}{E_h} \right) \) is the reduction function; \( L \) is length of the side of footing perpendicular to the
rotational axis, $\delta_p$ is friction angle between soil and structure, $h_p$, $T$ and $E_h$ are the lateral pressure, $\sigma'_{h}$ on footing side surfaces.

The Young’s modulus can be determined indirectly by oedometer test, 

$$E = \frac{E_{0.1} (1+\nu)(1-2\nu)}{1-\nu}$$  \hspace{1cm} (2.9)

where $E_{0.1}$ is the secant elastic modulus.

The ultimate overturning moment $M_u$ can be calculated with conventional bearing capacity approaches with some modification.

$$M_u = \frac{1}{R_M} M_f$$  \hspace{1cm} (2.10)

where $M_f$ is the failure overturning moment, $R_M = \frac{(\sigma_1-\sigma_3)_f}{(\sigma_1-\sigma_3)_u}$ =failure deviator/ultimate deviator, this value can be evaluated from triaxial tests plotted in a hyperbolic coordinate system (Duncan 1970).

Elastic response is considered at the earlier stage of footing stiffness study. A systematic compilation of the elastic stiffnesses of a rigid circular footing was done by Poulos and Davis (1974). Those that are related to this study are presented here.

For the rotational stiffness following Borowicka (1943) for a circular footing on an elastic half-space:

$$K_r = \frac{4ER^3}{3(1-\nu^2)}$$  \hspace{1cm} (2.11)

where R is radius of circular footing.
Even though the format is different, the above stiffness actually is the origin of stiffness equation (2.32) in SNAME (2002).

For a rigid circular footing on finite layer under moment loading (Yegorv 1961), the rotational stiffness is given by:

$$K_r = \frac{4R^3BE}{(1-\nu^2)} \quad (2.12)$$

where $B = \frac{1}{3} \alpha_1 + \frac{1}{5} \alpha_3$ and $\alpha_1, \alpha_3$ are tabulated coefficients (see Table. 2-1), $d$ is the depth of soil layer.

In this equation, Yegorv and Nitchiporovich (1961) introduced a new parameter $B$ to take into account the embedment effects, which can be regarded as an improvement to the previous method.

<table>
<thead>
<tr>
<th>$d/R$</th>
<th>$\alpha_1$</th>
<th>$\alpha_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>4.23</td>
<td>-2.33</td>
</tr>
<tr>
<td>0.5</td>
<td>2.14</td>
<td>-0.70</td>
</tr>
<tr>
<td>1.0</td>
<td>1.25</td>
<td>-0.10</td>
</tr>
<tr>
<td>1.5</td>
<td>1.10</td>
<td>-0.03</td>
</tr>
<tr>
<td>2.0</td>
<td>1.04</td>
<td>0</td>
</tr>
<tr>
<td>3.0</td>
<td>1.01</td>
<td>0</td>
</tr>
<tr>
<td>&gt;=5.0</td>
<td>1.00</td>
<td>0</td>
</tr>
</tbody>
</table>

For a rigid circular area on an elastic half-space under horizontal load (Muki 1961), the horizontal stiffness is given by:
\[ K_b = \frac{32(1-\nu)ED}{(7-8\nu)(1+\nu)} \]  

(2.13)

where \( E \) is elastic modulus of the soil, \( D \) is the diameter of the circle, and \( \nu \) is the Poisson’s ratio.

The inspiration to further study rotational stiffness is from soil-structure interaction analysis. However, previous researchers paid more emphasis on the response of column-bases and believed that the structural behavior might be improved by considering the rotational stiffness of the structure base itself (Hon 1987). Just as the past experience has shown, that could only partially contribute to the improvement in practice. Wiberg (1982) discussed the importance of footing stiffness in structural analysis. His work, based on numerical study, included two aspects: non-linear frame on elastic soil and non-linear frame on non-linear soil.

The soil stiffness is given as follows:

\[
\begin{bmatrix}
  s_{11} & s_{12} & s_{13} \\
  s_{21} & s_{22} & s_{23} \\
  s_{31} & s_{32} & s_{33}
\end{bmatrix}
\begin{bmatrix}
  u \\
  \theta \\
  w
\end{bmatrix}
= 
\begin{bmatrix}
  V \\
  M \\
  H
\end{bmatrix}
\]  

(2.14)

This equation can be simplified as a generalized spring: \( s \times n = N \)

where \( u \), \( \theta \), \( w \), and \( V \), \( M \), \( H \) are vertical, rotational, horizontal displacements and forces respectively acting on the footings, respectively.

For the case of linear soil with elastic, isotropic half-space properties, equation (2.14) can be represented by:

\[
\begin{bmatrix}
  s_{11} & 0 \\
  0 & s_{22}
\end{bmatrix}
\begin{bmatrix}
  u \\
  \theta
\end{bmatrix}
= 
\begin{bmatrix}
  V \\
  M
\end{bmatrix}
\]  

(2.15)
where

\[ s_{11} = 2\pi E / 3 \left( \ln 4x - 1/ \sqrt{1+1/4x^2} \right), \ x = d/b \]

\[ s_{22} = 4 - G (1 + 4\beta^2) b^3 / (\chi + 1), \ \chi = \frac{\lambda + 3G}{\lambda + G}, \]

\[ \lambda = \frac{E}{(1 + \nu)(1 - 2\nu)}, \ G = \frac{E}{2(1 - \nu)}, \ \beta = \frac{\ln x}{2\pi} \]

where \( d \) is embedment of footing, and \( b \) is width of footing.

For the non-linear soil case, a non-linear stiffness function is used to express the relationship of force and displacement.

\[
\begin{bmatrix}
    s_u(u, \theta) & 0 \\
    0 & s_v(u, \theta)
\end{bmatrix}
\begin{bmatrix}
    u \\
    \theta
\end{bmatrix} =
\begin{bmatrix}
    V \\
    M
\end{bmatrix}
\]

(2.16)

where \( s_u, s_v \) are non-linear spring stiffnesses; two approaches were recommended by Wiberg (1982) to obtain these two expressions.

The first approach is to introduce a yield surface into numerical modeling. Here an elliptical yield surface is represented by (see Fig. 2-3)

\[
\left( \frac{M}{M_y} \right)^2 + \left( \frac{N}{N_y} \right)^2 = 1
\]

(2.17)

The stiffness is based on a hyperbolic relation:

\[
\bar{X} = s_x
\]

(2.18)

\[
\bar{s} = \frac{1}{s_i + \frac{1}{x}}
\]

(2.19)

where \( \bar{X} \) and \( \bar{x} \) are normalized force and displacement respectively. The yield locus and force-displacement relation are represented in Fig. 2-3 and Fig. 2-4.

Another approach is through a parameter study in which \( V \) and \( M \) are obtained with fixed \( u/\theta \) ratio by finite element analysis.
Different plastic hinge assumptions were combined with clamped, hinged, pinned footings to analyze the response of these frames. Wiberg (1982) concluded that the soil stiffness significantly affects the loading capacity of the frame and the plastic hinge of the structure was also redistributed correspondingly. This is an early attempt to stress the importance of fixity assumptions of the footing on the structural performance.

Conventionally when designers analyze the soil-footing interaction, they do not consider the following effects: non-linearity of constitutive models of the soil, the effects of embedment, loading eccentricity, and stress level. Thinh (1984) conducted some experimental studies to incorporate these factors into a design method.

His work began with a series of experimental investigations. The testings were done in sand on strip, square, and rectangular footings separately. The soil parameters were obtained from direct simple shear tests and triaxial tests. From the experiments he determined the moment-rotation and load-displacement relationships. Based on the test data and regression analysis, the rotational stiffness was given by a new hyperbolic function as follows.

\[
\frac{X}{eQ} = b_0 + b_1 X
\]

where \( X = 10^{-3} \alpha \); \( 1/(10^{-3}b_0) \) is the initial rotation stiffness, \( b_0=1000S_{ri} \), \( S_{ri} \) is the initial rotational stiffness; \( 1/b_1 \) is the ultimate overturning moment; \( eQ_{in}, e \) is the eccentricity in m, \( Q \) is the vertical load in kN; \( \alpha \) is the angle of rotation in radian
corresponding to the moment eQ. The non-linear relationship is reflected in Fig. 2-5.

Through the transformation of equation (2.20), the secant stiffness and tangent stiffness can be obtained as follows.

Secant rotational stiffness,
\[ S_{\alpha} = \frac{eQ}{\alpha} = S_{ni} \frac{1}{1 + \frac{S_{ni}}{eQ}} \]  
(2.21)

Tangent rotational stiffness,
\[ S_{\alpha} = \frac{d(eQ)}{d(\alpha)} = S_{ni} \frac{1}{\left(1 + \frac{S_{ni}}{eQ}\right)^2} \]  
(2.22)

During the analysis of the test data, Thinh also studied appropriate initial rotational stiffness \( S_{ni} \). Some of the rotational stiffness vs. moment are plotted in Fig. 2-6 where the eccentricity of the vertical load is 0.125 times of the footing width, the non-linear relationship of rotational stiffness and overturning moment under three embedment ratio, \( d/b = 0, 1, 2 \), is presented.

Xiong et.al (1989) tested a series of rectangular footings under static lateral loading and obtained the overturning resistance in different soils. Their tests were conducted on the surface foundations and embedded foundations. Because the contact stress on the foundation is unknown, the authors assumed three different stress distributions, which are bilinear, curved, and linear, for the bottom reaction and side surface reaction. Based on the assumptions and the test results, the ultimate vertical and moment resistance of the footings were expressed with suitable known parameters. In this model, the soil reaction was simulated with
elastic springs (Fig. 2-7). The bottom spring subgrade reaction modulus is given by:

$$k_b = \frac{5.34G}{(1-\nu)\sqrt{lb}}$$  \hspace{1cm} (2.23)

And the single side spring subgrade reaction modulus is shown as:

$$k_s = \frac{(1/1.3)5.34G}{(1-\nu)\sqrt{hl}}$$  \hspace{1cm} (2.24)

where l, b, h are the length, width and height of the footing respectively.

The shear modulus of the soil is modified by a function of the uplift ratio r, $G=Gs f(r)$

with the function $f(r)$ obtained by fitting test data with uplift ratio

$$f(r) = 1 - 0.9r + 0.1\sin(2.5\pi r)$$  \hspace{1cm} (2.25)

where $G_s$ is initial shear modulus. $r$ is the uplift ratio, $r = (b - \overline{b})/b$, $\overline{b}$ is the width of the footing where the soil is in direct compression.

Even though the authors analyzed the tested model and found their model could fit the test data very well, the model could not be applied to other cases easily. What is more, the model is based on elastic modulus and may not reflect the soil behavior correctly. But it can provide a good example to analyze the soil response of footing under lateral loading, considering both the side and the bottom of the footing surface.

Inspired by previous studies, Melchers (1992) did some tests on full scale footings applying combined vertical, horizontal, and moment forces on them. Following Xiong et.al(1989), he also carefully observed the uplift effects and the side surface
influence on the footing stiffness. A three-item equation was deduced to represent
the rotational moment as follows:

\[ M_b = K_T \theta = K_b \theta + (K_s \Delta) h_e + \sum_i I W_i \]  \hspace{1cm} (2.26)

where \( K_T \) is total rotational stiffness, \( K_b \) is the rotational stiffness for the base, \( K_s \) translational stiffness of the sides, \( \Delta \) horizontal translation of the side at height \( h_e \),
the relevant lever arm. The last term represents the sum of shearing forces
response around the vertical sides of the footing.

He used \( K_b \) and \( K_s \) value from Poulos and Davis (1974). The rotational stiffness is
given by:

\[ K_b = \frac{b^2 l E_b}{(1 - \nu^2) I_\theta} \]  \hspace{1cm} (2.27)

where \( I_\theta = \frac{16}{\pi \left(1 + 0.22 \left(\frac{b}{l}\right)\right)} \); \( E_b \) is the soil elastic modulus at the footing base, \( \nu \)
poisson’s ratio, and \( b,l \) are the width and length of the footing. The translational
stiffness is given by:

\[ K_s = \alpha m E_s \sqrt{h l} \]  \hspace{1cm} (2.28)

In this model, the author considered the uplift effects under bending moment by
introducing an effective base breadth as follow:

\[ b_e = b \left[1.5 - 0.5 \left(\frac{M_b}{M_u}\right)\right] \]  \hspace{1cm} (2.29)

The iteration will be applied to adjust the variation of \( K_b \) resulting from the
effective base breadth variation. Although this is an improvement as it considers
the non-linear effect slightly, it is only a semi-empirical analytical approach.
These stiffness studies can be generalized as linear or non-linear stiffness studies, on rectangular or circular onshore footings. Even though they provide a simple and reasonable way to account for some parameters, such as footing geometry, side wall effects etc, they have yet to capture the complete behavior of the spudcan.

### 2.2.2 Soil stiffness in SNAME (2002)

SNAME (2002) recommended that the rotational, vertical and horizontal stiffness of the soil to be simulated as linear springs and applied to the spudcan when site assessment is performed.

**Vertical stiffness:**

\[
K_v = \frac{2G_v D}{(1-\nu)}
\]  
(2.30)

**Horizontal stiffness:**

\[
K_h = \frac{16G_h D (1-\nu)}{(7-8\nu)}
\]  
(2.31)

**Rotational stiffness:**

\[
K_r = \frac{G_r D^3}{3(1-\nu)}
\]  
(2.32)

where D is the equivalent footing diameter, \( \nu \) is the soil Poisson’s ratio, \( G_v, G_h \), and \( G_r \) are vertical, horizontal, rotational shear modulus of the soil respectively.

The estimation of shear modulus \( G \) is empirically given by following equations for clay and sand respectively.
In clay, the rigidity index $I_r$ is given as follows:

$$
G/C_u= \begin{cases} 
50 & \text{OCR}>10 \\
100 & 4<\text{OCR}<10 \\
200 & \text{OCR}<4
\end{cases}
$$

(2.33)

The shear modulus of the clay for the vertical, horizontal and rotational stiffness are assumed to be the same.

If $\text{OCR}>4$ and the soil is susceptible to cyclic degradation, the calculated rotational stiffness should be reduced by a factor of 1.25.

In dense sand, the shear moduli are given by following equations,

$$G_v=36600+24.9(V_{Lo}/A)$$

(2.34)

$$G_h=1100+5.6(V_{Lo}/A)$$

(2.35)

$$G_r=4100+11.5(V_{Lo}/A)$$

(2.36)

where the unit is in kN/m$^2$, $V_{Lo}$, $A$ are the ultimate bearing capacity and maximum area of the spudcan, respectively.

When loose sand is encountered, the following factor should be used to deduce the corresponding shear modulus,

$$f(e) = \frac{\left(2.973 - e\right)^2}{1+e}$$

(2.37)

$$G_{\text{loose}} = G_{\text{dense}} \left| \frac{f(e_L)}{f(e_D)} \right|$$

(2.38)

where $f(e_L)$=value of $f(e)$ for loose sand, $f(e_D)$=value of $f(e)$ for dense sand.

Studies have shown that the embedment of spudcan has an influence on the stiffness. The vertical, horizontal, rotational spring stiffness can be multiplied by the following depth factor $K_{d1}$, $K_{d2}$, $K_{d3}$, respectively to account for the
embedment effects. (see Table. 2-2, where d and R are embedment depth and radius of the spudcan, respectively)

Table. 2-2: Embedment factors in foundation stiffness (SNAME 2002)

<table>
<thead>
<tr>
<th>d/R</th>
<th>K_{d11}</th>
<th>K_{d12}</th>
<th>K_{d21}</th>
<th>K_{d22}</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Case 1</td>
<td>Case 2</td>
<td>Case 1</td>
<td>Case 2</td>
</tr>
<tr>
<td>0.5</td>
<td>1.15</td>
<td>1.21</td>
<td>1.33</td>
<td>1.49</td>
</tr>
<tr>
<td>1.0</td>
<td>1.28</td>
<td>1.41</td>
<td>1.44</td>
<td>1.71</td>
</tr>
<tr>
<td>2.0</td>
<td>1.42</td>
<td>1.70</td>
<td>1.51</td>
<td>1.92</td>
</tr>
<tr>
<td>4.0</td>
<td>1.59</td>
<td>2.00</td>
<td>1.61</td>
<td>2.06</td>
</tr>
</tbody>
</table>

In Table. 2-2, case 1 and case 2 represent the case without backflow and with backflow, respectively.

### 2.2.3 Finite element study on footing stiffness

The above-mentioned stiffness studies are mainly analytical or semi-empirical solutions. Bell (1991) carried out a systematic study on footing stiffness with the finite element method. His analysis included parameters, such as backflow, embedment and soil Poisson’s ratio.

The studies done by Bell (1991) and Ngo-Tran (1996) indicated that the horizontal and rotational displacements were cross-coupled. The incremental elastic relationship can be expressed as

\[
\begin{bmatrix}
\frac{dV}{dM/2R} \\
\frac{dH}{dV}
\end{bmatrix} = 2GR
\begin{bmatrix}
k_1 & 0 & 0 \\
0 & k_2 & k_4 \\
0 & k_4 & k_3
\end{bmatrix}
\begin{bmatrix}
dw^e \\
2Rd\theta^e \\
du^e
\end{bmatrix}
\]

where \(dV, dM, dH\) are incremental vertical load, moment, and horizontal load, respectively; \(dw^e, d\theta^e, du^e\) are incremental elastic vertical, rotational, horizontal displacement, respectively; non-dimensional stiffness value \(k_1, k_2, k_3, k_4\) can be
found in Table. 2-3 in which case 1, 2, 3 are footing without backflow, footing with backflow and full sidewall interaction respectively (see Fig. 2-8), and \( Z_d \) is the embedment depth of the footing.

Table. 2-3: Non-dimensional soil stiffness factors (Bell 1991)

<table>
<thead>
<tr>
<th>Poisson's ratio</th>
<th>Elastic footing results for three cases of embedment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Case</td>
</tr>
<tr>
<td>( v=0.0 )</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>2</td>
</tr>
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</table>
The stiffness matrix may be inverted to become the flexibility matrix to ease the calculation if the combined loads are known.

\[
\begin{bmatrix}
\frac{F_1}{GR} & 0 & 0 \\
0 & \frac{F_2}{GR} & \frac{F_4}{GR} \\
0 & \frac{F_4}{GR^3} & \frac{F_3}{GR^3}
\end{bmatrix}
\begin{bmatrix}
V \\
H \\
M
\end{bmatrix} = \begin{bmatrix}
u \\
w \\
\theta
\end{bmatrix}
\]

(2.40)

where the flexibility parameters can be expressed in terms of the stiffness parameters by the following relationships:

\[F_1 = \frac{1}{k_1}\]

(2.41)

\[F_2 = \frac{k_3}{k_2k_3 - k_4^2}\]

(2.42)

\[F_3 = \frac{k_3}{k_2k_3 - k_4^2}\]

(2.43)
The \( k \) values deduced by Bell (1991) are listed in the Table. 2-3.

Some other researchers carried out studies to obtain the shear modulus for different soils. For clay, the soil shear modulus is recommended by Martin (1994) as,

\[
G = I_r s_u
\]  

(2.45)

where \( s_u \) is the undrained shear strength measured at 0.15 diameter below the reference point of the spudcan (the maximum area location of spudcan). \( I_r \) is the rigidity index. On clay it can be taken as

\[
I_r = \frac{G}{s_u} = \frac{600}{OCR^{0.25}}
\]  

(2.46)

For sand, Cassidy (1999) recommended that the shear modulus can be obtained by

\[
\frac{G}{p_a} = g \left( \frac{V}{4p_a} \right)^{0.5}
\]  

(2.47)

where \( V \) is the spudcan vertical load, \( A \) the spudcan area and \( p_a \) atmospheric pressure. Dimensionless constant \( g \) can be determined from

\[
g = 230 \left( 0.9 + \frac{D_R}{500} \right)
\]  

(2.48)

where \( D_R \) is relative density of sand.

### 2.3 Yield surface

The state of the art on yield surface of spudcan under combined loads is discussed
in this section. SNAME (2002) recommended a yield surface which can depict the spudcan response under different load combinations. Two other numerical models, namely, Van Langen’s model and Houlsby & Martin’s model were reviewed. Houlsby & Martin’s model is semi-empirically created from small-scale experimental studies. The results of physical modeling with 1-leg or 3-leg spudcan are reviewed in this section.

2.3.1 Yield interaction in SNAME (2002)

At a specified depth, the soil cannot resist the combined loading infinitely. The envelope of the allowable combined loadings represents the yield surface for this specific depth. Within this yield surface, the spudcan will response linearly with the stiffness value according to equations (2.30)-(2.32). Once the combined loads touch the yield surface, the soil yields. Thereafter, there are three possibilities. First, the spudcan may be linearly unloaded with the above-mentioned stiffnesses such that the combined loads will lie within the yield surface. Second, the combined loads may still lie on the yield surface, but once the load combination is changed, the corresponding plastic displacements will vary according to the relevant flow rule. Third, the combined loads may go beyond the current yield surface, leading to hardening behavior and the spudcan will penetrate further.

The equation for the yield surface in SNAME(2002) is:

\[
\frac{H}{H_{L0}}^2 + \left(\frac{M}{M_{L0}}\right)^2 - 16 \left(\frac{V}{V_{L0}}\right)^2 \left[1 - \frac{V}{V_{L0}}\right]^2 = 0
\]

(2.49)

For sand:
\[ H_{Lo} = \left( \frac{C_1}{C_2} \right) \left( \frac{V_{Lo}}{4} \right) = 0.12V_{Lo} \]

\[ M_{Lo} = C_1V_{Lo}B/4 = 0.075V_{Lo}B \]

where \( C_1 = 0.3, \ C_2 = 0.625 \)

For clay:

\[ H_{Lo} = c_{wu}A + (c_{wu} + c_{ul})A_s \]

\[ M_{Lo} = 0.1V_{Lo}B \]

where \( c_{wu} \) is undrained cohesive shear strength at maximum bearing area, \( c_{ul} \) is undrained shear strength at spudcan tip, \( B \) is effective spudcan diameter, \( A \) is the maximum bearing area of the spudcan, and \( A_s \) is the spudcan laterally projected area.

The associated-flow rule is assumed in SNAME (2002), so the potential surface is the same as the yield surface. The hardening (softening) is governed by the ultimate vertical bearing capacity, \( V_{Lo} \) at the specific depth which will be discussed in the next section.

The estimation of the rotational stiffness of the spudcan will follow the below-mentioned procedures. Firstly, an estimate from equation (2.32) is taken as the initial input of the analysis. If the left side of equation (2.49) is larger than 0, the combination of loading lies outside the yield surface. The rotational stiffness of the spudcan must be reduced until the load combination lies on the yield surface. The reduction factor is arbitrary and the analysis needs to be iterated.

If the left side of equation is less than 0, the combination will lie inside the yield surface. The initial \( K_r \) should be reduced by a factor \( f_r \).
\[ f_r = \sqrt{(1-r_f)} + 0.1e^{\frac{100(r_f-1)}{r_f}} \]  

(2.50)

\[ r_f = \left\{ \left[ \frac{H}{H_{L0}} \right]^2 + \left[ \frac{M}{M_{L0}} \right]^2 \right\}^{0.5} \]  

(2.51)

where \( r_f \) is failure ratio, measuring the proximity to yield for generalized combined loads; if \( r_f \geq 1.0 \), it means the combined loads lie outside yield surface. Under such condition, the reduction factor is not applicable. \( f_r \) represents the ratio of rotational stiffness to its maximum value.

Recently some researchers have challenged this definition and indicated that “the meaning of the equation is unclear” (Templeton 2007). A unified and generalized reduction factor was introduced by Templeton.

### 2.3.2 Physical modeling relevant to yield surface study

#### 2.3.2.1 Single leg spudcan

By the end of 1980’s, few researchers have carried out spudcan model tests. An important enhancement is done by Tan (1990) who compared the unloading-reloading process with unloading-reloading behavior in the Modified Cam Clay model. Meanwhile, he firstly developed the concept of “sidewipe tests” to determine the yield surface, even though his original objective was to find the relationship of horizontal displacement and load when the vertical penetration was kept constant. His single-leg conical footing was tested in the
Cambridge centrifuge. Through the results, Tan extended the understanding of plasticity behavior of spudcan under combined loadings.

Following Tan’s work, Martin (1994) advanced the plasticity model. His work was based on more than 30 tests with independent loading system under 1g condition in Oxford University. This creative loading system is shown in Fig. 2-13. In his study, a new type of load cell which can measure vertical, horizontal, moment forces simultaneously was also introduced. Three types of tests, probing tests, tracking tests and looping tests, were carried out to experimentally investigate the shape of the yield surface under combined loads. The probing test is defined as the test method to find a yield point under constant vertical load. Tracking test, namely, sideswipe test, is the test to find the yield locus under constant vertical penetration when combined loads are applied on the spudcan. Looping test is the test to detect the cross section of yield surface under constant vertical load. The schematic plots are shown in Fig. 2-14~Fig. 2-16. Based on these tests, Martin experimentally established the shape of yield surface of the spudcan under combined loads. At the same time, he investigated the non-associated flow rule in the V-H and V-M plane, and derived the mathematical formulation of plasticity model for spudcan in clay.

After that, Gottardi (1999) utilized the same apparatus to conduct a series of tests on sand. Based on Gottardi’s data, Cassidy (1999) conducted a series of analysis. He found that the yield surface and potential surface were no longer identical for this plasticity model in sand. The yield surface follows different locus from the
one on clay. Using plasticity theory, he deduced the plasticity model of force-resultant for spudcan in sand, composed a generalized hardening law, independent yield and potential surfaces, and non-associated flow rule. These two strain-hardening models will be elaborated in Section 2.3.3.2.

### 2.3.2.2 Three legs jack-up platform

In the beginning of 1990’s, a joint industry study was conducted in the Cambridge drum centrifuge Murff et al (1991). Part of the studies was dealing with spudcan fixity. The models of three single spudcan and one 3-leg jack up unit were tested in Leighton Buzzard sand. The relationship of load and displacement, the yield interaction of vertical and horizontal force were illustrated. Meanwhile, simple comparisons of analyzed secant rotational stiffness and the one obtained from linear spring condition were made. In the middle of 1990’s, similar jack up models were tested on drained and partially drained sand in the same laboratory(Dean 1996). The variation of the excess pore pressure under cyclic loading with utilization of the skirts was carefully monitored. It is concluded that the vertical displacement will benefit the reduction of potential liquefaction under cyclic loadings and fixity will be reduced following the increase of cyclic load amplitudes. Almost at the same time another set of spudcan models in loose sand was tested by Tempeton (1997) in Cambridge. One of the significant findings in this test is a summary of lower bound value of rotational stiffness as $K_{rot}/V_{L0}D=30$
where $V_{L0}$ is bearing capacity of the soil, and $D$ is the spudcan diameter at maximum area. This conclusion coincided with the recommendation generalized from later field measurements by some researchers.

Temperton also found that plasticity occurred even before the combined loading reaches the yield surface. Displacement check should be accounted for even before the soil yield. In addition, the plastic deformations under cyclic loading within yield surface envelope and the asymmetry of windward and leeward legs also were addressed with respect to load path.

A series of spudcan models including single or 3 legs spudcan were built in the University of Western Australia by Vlahos et.al(2004; Vlahos 2006). These models simulate real F&G jack up units and tested in soft clay (see Fig. 2-17). Being different from earlier experiments in Oxford University, these tests were instrumented mainly on the combined loading and displacement of hull and spudcan. The orientation of lateral load and jack up leg length were the main parameters investigated. The jack-up responses under pushover loads were recorded and the load path of windward/leeward leg and spudcan were analyzed, using Martin’s plasticity model. The comparison between tests results and analysis results based on different spudcan fixity clearly shows that the hardening force resultant model is more accurate (Cassidy 2007). As can been seen in Fig. 2-18, lateral load and displacement of the hull using the plasticity model can match well with experimental data, even though the soil stiffness affect the results to some extent, while the fixity simulated with pinned,
encastred, and linear spring do not agree well with the test data. The following load path plot of jack up 3 legs also verified the efficiency of this model. In Fig. 2-19, the strain-hardening model C derived by Cassidy (refer to Section 2.3.3.2) once again displayed its advantage beyond existing linear spring method. Vlahos (2004) concluded that this plasticity model could “retrospectively simulate the general trends of the experimental data”. Another contribution by Vlahos is the application of 1D hyper plasticity model which could simulate the cyclic behavior of spudcan under combined loads. One typical simulated result is shown in Fig. 2-20. Even though simple, it is the first time that the hyper plasticity model is incorporated into experimental study.

2.3.3 Other yield surface theories

The soil response beneath the spudcan under combined loads is simulated as soil spring at the beginning stage of fixity study. The researchers found that once the displacements of the spudcan are large, the spudcan will display plasticity behavior. The interaction expression in equation (2.49) actually belongs to one of the plasticity models. Below another two plasticity models will be reviewed.

2.3.3.1 Van Langen(1993) model

Based on the hardening plasticity theory, Van Langen (1993) developed a model to predict the behavior of jack up foundation. This model overcomes the shortcomings of step 3 presented in Section 2.4, that is neglecting spudcan failure
of both sliding and bearing.

The failure function of spudcan under combined loading is expressed as:

\[
f_u = \left( \frac{M}{M_0} \right)^2 + \left( \frac{H}{H_0} \right)^2 - c_1 \left( \frac{V}{V_0} \right) \left( 1 - \left( \frac{V}{V_0} \right)^2 \right)
\]  

(2.52)

For clay, \(c_1=4.0;\) \(c_2=1.0\)

\[H_0 = s_u A + 2s_u A_h, M_0 = V_o D/3 \pi ;\]

For sand, \(c_1=0.5;\) \(c_2=1.0\)

where \(H_0 = V_o, M_0 = V_o D\)

\(s_u,\) undrained shear strength; \(A,\) maximum plan area of embedded spudcan section; \(A_h,\) spudcan laterally projected embedded area of spudcan; and \(D,\)

spudcan diameter.

To represent the relationship of bearing capacity and vertical plastic displacement, the following function is defined,

\[V_0 = V_0 \left( V_p \right)
\]  

(2.53)

However, the properties of soil are significantly different from metal material and they exhibit non-linear response even far before combined loadings reach the yield surface. Thus, an inner yield function \(f_e\) is defined as follows. For \(f_e(V,H,M)<0,\) the plastic deformations can be neglected. If \(f_e>0\) and \(f_0<0,\) the rotational stiffness becomes non-linear, but foundation failure does not occur.

\[
f_e = \sqrt{\left( \frac{M}{c_3 M_0} \right)^2 + \left( \frac{H}{H_0} \right)^2 - c_1 \left( \frac{V}{V_0} \right) \left( 1 - \left( \frac{V}{V_0} \right)^2 \right)}
\]  

(2.54)

where the parameter \(c_3\) is a symptom of non-linearity onset, recommended as 0.3 for sand and 0.5 for clay.
The onset of failure can be judged by a parameter \( \bar{\theta}_p \), for which

\[
f(V, H, M, u_p, v_p, \theta_p) = \begin{cases} f_c \text{for} \theta_p \geq \bar{\theta}_p \\ f_c \text{for} \theta_p = 0 \end{cases}
\]  

(2.55)

Through interpolation, the intermediate value can be obtained as

\[ f = f_c + (f_u - f_c)G \]  

(2.56)

where the hardening function \( G = G(\theta_p, \bar{\theta}_p) \) satisfies the following conditions:

\[ G = 0 \text{ for } \theta_p = 0 \]

\[ 0 < G < 1 \text{ for } 0 < \theta_p < \bar{\theta}_p \]

\[ G = 1 \text{ for } \theta_p \geq \bar{\theta}_p \]

The smooth transition can be obtained from

\[ G = \sqrt{1 - \left(1 - \frac{\theta_p}{\bar{\theta}_p}\right)^2} \]  

(2.57)

The onset of failure can be defined as:

\[ \bar{\theta}_p = c_4 \frac{V_o D}{k_o^e} \]  

(2.58)

where \( k_o^e \) is the spudcan elastic rotational stiffness.

Experimental study showed that \( c_4 \) can be taken as 1.4 for sand and 1.0 for clay.

Using the same principle the displacement of the spudcan can be decomposed into elastic and plastic parts:

\[ \ddot{u} = \ddot{u}_e + \ddot{u}_p \]  

(2.59)

where \( \ddot{u}^T = (\ddot{u}, \dot{v}, \dot{\theta}) \)

where a superimposed dot indicates an increment.

\[ \dot{Q} = K_o \dot{u} \]  

(2.60)
where \( \dot{Q}^t = (H, V, M) \) and \( K_e = \begin{bmatrix} k_h^e & k_v^e & k_\theta^e \end{bmatrix} \)

where \( k_h^e, k_v^e, k_\theta^e \) are the elastic horizontal, vertical and rotational stiffness respectively. Following classical plasticity theory, the plastic deformation is derived from a flow rule.

\[
\dot{\ell}_p = \dot{\lambda} \frac{\delta g}{\delta Q}
\]

which involves a plastic potential function \( g \):

\[
g = \begin{cases} f & \text{for clay} \\ g_e + (g_u - g_e)G & \text{for sand} \end{cases}
\]

where

\[
f = \text{yield function}
\]

\[
g_u = \sqrt{\left(\frac{M}{M_0}\right)^2 + \left(\frac{H}{H_0}\right)^2 - \frac{2}{3} c_1 \left(\frac{V}{V_0}\right)^2 \left[1 - \left(\frac{V}{V_0}\right)^2\right]}
\]

\[
g_e = \sqrt{\left(\frac{M}{c_2 M_0}\right)^2 + \left(\frac{H}{H_0}\right)^2 - \frac{2}{3} c_1 \left(\frac{V}{V_0}\right)^2 \left[1 - \left(\frac{V}{V_0}\right)^2\right]}
\]

\( G = \text{hardening function} \)

The elastic stiffness values of equation are derived from

\[
k_h^e = \frac{2G_D}{1 - \nu}; k_v^e = \frac{16G_D}{7 - 8\nu}; k_\theta^e = \frac{G_D}{3(1 - \nu)}
\]

Compared with the results of centrifuge tests, this model indicated good fit to the available data.

### 2.3.3.2 Strain-hardening plasticity model

Since the idea of linking the shape of V, H, M interaction diagram with the
displacements of the footings was introduced by Butterfield (1978), it has been greatly enhanced, especially by the researchers from Oxford University, such as Houlsby and Martin (1994), and Cassidy (1999). Schotman (1989) firstly utilized the theory in the spudcan numerical modeling. He suggests that the use existing plasticity theory to model combined load-displacement behavior of spudcan was preliminary and further detailed study was needed to verify and improve the modeling technology. Houlsby et.al (2004; 1994) did systematic studies on this method. Today, this model has advanced from 3 degrees of freedom to 6 degrees of freedom (Bienen 2006). Corresponding program has also been developed to facilitate the spudcan-soil analysis combined with structure models. A large number of experiments on sand and clay (Bienen 2006; Cassidy 1999; Martin 1994; Vlahos 2004) have been carried out to calibrate the model.

The model on the basis of experimental calibration consists of four parts. The elasticity part has been elaborated in Section 2.2.3. The other three parts will be briefly described here.

### 2.3.3.2.1 Yield surface

The yield surfaces of spudcan on clay and on sand were developed by Martin (1994) and Cassidy (1999) respectively. They were generalized as one equation by Cassidy (2005).

\[
f = \left( \frac{H}{h_0 V_{L0}} \right)^2 + \left( \frac{M / 2R}{m_0 V_{L0}} \right)^2 - \frac{2aHM / 2R}{h_0 m_0 V_{L0}^2} - \left[ \left( \frac{\beta_1 + \beta_2}{\beta_1 \beta_2} \right)^{\beta_1 + \beta_2} \right] \left( \frac{V}{V_{L0}} \right)^{2\beta_1} \left( 1 - \frac{V}{V_{L0}} \right)^{2\beta_2} = 0
\]

(2.66)
where $V_{L0}$ determines the size of the yield surface and indicates the bearing capacity of the foundation under purely vertical loading. Furthermore, $V_{L0}$ is governed by the vertical plastic penetration and is determined from the strain-hardening law. The dimensions of the yield surface in the horizontal and moment directions are determined by $h_0$ and $m_0$ respectively and accounted for eccentricity (rotation of the elliptical cross-section) in the $M/2R:H$ plane. The parameters $\beta_1$ and $\beta_2$ round off the points of surface near $V/V_{L0}=0$ and $V/V_{L0}=1$. Appropriate yield surface parameters are listed in Table. 2-4 (Randolph 2005).

<table>
<thead>
<tr>
<th>Clay (Martin and Houlsby)</th>
<th>Dense silica sand (Houlsby and Cassidy)</th>
<th>Loose carbonate sand(Cassidy et al)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$h_0$</td>
<td>0.127</td>
<td>0.116</td>
</tr>
<tr>
<td>$m_0$</td>
<td>0.083</td>
<td>0.086</td>
</tr>
<tr>
<td>$\beta_1$</td>
<td>0.764</td>
<td>0.9</td>
</tr>
<tr>
<td>$\beta_2$</td>
<td>0.882</td>
<td>0.99</td>
</tr>
</tbody>
</table>

In clay, $\alpha$ varies with $V/V_{L0}$ and takes the form

$$a = e_1 + e_2 \left( \frac{V}{V_{L0}} \right) \left( \frac{V}{V_{L0}} - 1 \right)$$

(2.67)

$e_1=0.518$ and $e_2=1.180$ are recommended.

### 2.3.3.2.2 Hardening law

The variation of $V_{L0}$ with plastic vertical displacement $w_p$ defining a hardening law and the size of the yield surface can be determined either by constructing curves based on bearing capacity theory or experimental evidence or both, as will
be elaborated in Chapter 6.

\[
V_{L0} = \frac{\left(1 - f_p\right)\left(kw_p \frac{V_{0m}}{w_{pm}}\right) + f_p \left(\frac{w_p}{w_{pm}}\right)^2}{\left(1 - f_p\right)\left(1 - \left(2 - \frac{kw_{pm}}{V_{0m}}\right)\right) + \left(\frac{w_p}{w_{pm}}\right)^2} V_{0m}
\]  

(2.68)

where \(k\) is the initial plastic stiffness, \(f_p\), a dimensionless constant that describes the limiting magnitude of vertical load, \(V_{0m}\) is the peak value of \(V_{L0}\), and \(w_{pm}\) the value of plastic vertical penetration at this peak.

### 2.3.3.2.3 Flow rule

The yield surface expansion (or contraction) is determined by hardening law. The ratios of plastic displacements are governed by the flow rule. The simplest form is associated flow, the ratios are determined by

\[
dw_p = \lambda \frac{\partial f}{\partial V}, du_{3p} = \lambda \frac{\partial f}{\partial H_3}, d\theta_{2p} = \lambda \frac{\partial f}{\partial M_2},
\]

(2.69)

where \(\lambda\) is a non-negative multiplier that can be determined from the requirement that an elastic-plastic load step must remain on the yield surface. Both the clay and sand experiments (Cassidy 1999; Martin 1994) showed associated flow only in the M/2R:H plane. Martin developed a simple empirical modification of the vertical component in clay with non-associated flow rule,

\[
dw_p = \zeta \lambda \frac{\partial f}{\partial V}, \quad \text{here} \quad \zeta = 0.6.
\]

For sand, a plastic potential was developed by Cassidy (1999). Its expression is similar to that of the yield surface, but shape and size are scaled by two associated factors \((a_h, a_m)\).
where $\beta_1, \beta_4$ are curvature factors of plastic potential at low stress and high stress respectively, $a_h$ and $a_m$ are the association factors of horizontal force and moment.

### 2.4 Moment fixity consideration in SNAME(2002)

There are three ways to take account of the rotational stiffness in the response analysis. Fig. 2-21 shows a flowchart on calculating spudcan response taking into account of fixity,

a). Using quasi-static analysis iteratively to derive the rotational and horizontal stiffness with loadings obtained from linear spring foundation case which initially are obtained using equation (2.30)–(2.32).

b). Firstly conducting structural dynamic analysis in single degree of freedom or multi degrees of freedom with soil springs. Second, carrying out a final quasi-static structural analysis with linear fixity using the procedures stated in a).

c). Analyzing both dynamic response of the structure and seabed reactions with consideration of the effects of foundation fixity.

A simple approach to account for foundation fixity in method a) is briefly described as follows. In pseudo-static analysis the above-mentioned linear springs are added to the model and hence produce the moment load on the spudcan-leg. Confined by yield interaction equation, rotational stiffness is adjusted to fit the
yield surface. The approach is as follows:

1) Applying dead, live, environmental, inertial loads to the structural model of jack-up with vertical, horizontal, rotational springs applied to spudcan.

2) Evaluate the status of the combined forces at each spudcan. The soil properties at the designated site will be used to estimate $V_{Lo}$, $H_{Lo}$, $M_{Lo}$ using equation (2.49). If the force combination lies outside the yield surface under extreme loading, the rotational spring stiffness would be reduced.

3) The above process is iterated until the force combinations lie essentially on the yield surface. If the moment tends to be zero at this time, while the force combination is still outside the yield surface, a bearing failure is indicated. The spudcan will penetrate into the stiffer layer, as represented by the hardening rule in numerical analysis.

4) If the force combination initially falls within the yield surface, rotational stiffness must be adjusted based on the above mentioned reduction factor $f_r$ and yield interaction function.

### 2.5 Ultimate bearing capacity

In the strain-hardening force resultant model, the size of the yield surface is governed by the ultimate bearing capacity of the spudcan, $V_{Lo}$. The hardening rule is also expressed as a relationship of ultimate bearing capacity and plastic displacement. Hence, the study of ultimate bearing capacity will play a
significant role in spudcan fixity analysis.

In this section, three bearing capacity theories on clay are discussed, namely, SNAME (2002), API-RP2A (2002) and Houlsby&Martin (2003). The results analyzed with these methods will be compared with test data in latter sections and a theory among them will be selected. The subsequent partial drained analysis will be based on this.

2.5.1 SNAME (2002)

This method recommended by SNAME (2002) is based on Skempton’s undrained bearing capacity theory. For clay,

\[ V_{Lo} = (c_u N_s d_c + p_o') A \]  

(2.71)

where \( p_o' \) is effective overburden pressure at the maximum area of spudcan.

Meyerhof’s backflow criterion (Meyerhof 1972) has been applied to the calculation of ultimate bearing capacity. When backflow occurs, the ultimate vertical bearing capacity will be given by

\[ V_{Lo} = (c_u N_s d_c + p_o') A - F_o A + \gamma V \]  

(2.72)

The leeward and windward legs will display different responses under combined loads. SNAME (2002) provided the following recommendations to check this direction effect.

Leeward leg should be checked according to the following equation.

\[ Q_{VH} = \gamma_1 VH_D + \gamma_2 VH_L + \gamma_3 (VH_E + \gamma_4 VH_{Dn}) \]  

(2.73)

where \( \gamma_1 = 1.0; \gamma_2 = 1.0; \gamma_3 = 1.15; \gamma_4 = 1.0 \)
VH₀: vector of vertical and horizontal leg reaction due to the weight of the structure and non-varying loads including:

- Weight in air including appropriate solid ballast
- Equipment or other objects.
- Buoyancy
- Permanent enclosed liquid.

VHₐ: vector of vertical and horizontal leg reaction due to maximum variable load positioned at the most onerous center of gravity location applicable to extreme conditions.

VHₑ: vector of vertical and horizontal extreme leg reaction due to the assessment return period wind, wave and current conditions.

VH₀n: vector of vertical and horizontal leg reaction due to the inertial loadset which represents the contribution of dynamics over quasi-static response.

\[ Q_{VH} \leq \phi_{VH} F_{VH} \]  \hspace{1cm} (2.74)

where: FᵥH: vertical leg reaction during preloading, and \( \phi_{VH} \): Resistance factor for foundation capacity during preload.

The combination of vertical and horizontal loading can be solved by the following equation.

In sand,

\[
F_{VH} = A \left\{ 0.5 \gamma' B N_{γ'} s_p d_e \left[ 1 - \left( \frac{F_H}{F_{VH}} \right)^{s/m+1} \right] \right\} + A p_{o N} N_{p} s_d \left[ 1 - \left( \frac{F_H}{F_{VH}} \right)^{m} \right] \]  \hspace{1cm} (2.75)

For different \( F_H/F_{VH} \), corrected horizontal capacity is given by:

\[
F_H = F_H^* + 0.5 \gamma' \left( k_p - k_a \right) \left( h_1 + h_2 \right) A_j \]  \hspace{1cm} (2.76)
Iteration will continue until convergence occurs.

In clay,

\[ F_{VII} = A \left[ N_c c_s \left[ 1 - \left( 1.5 F_{H*}^* / N_c A c_s \right) \right] + p_v N_q s_q \left( 1 - F_{H*}^* / F_{VII} \right)^{\frac{1}{5}} d_q \right] \]  

(2.77)

Substituting the value of \( F_{VII} \) and solving for \( F_{H*}^* \), \( F_H \) can be given by:

\[ F_{H*} = F_{H*}^* + \left( c_{mo} + c_{md} \right) A_e \]  

(2.78)

Windward legs should be checked following the sliding check procedures.

### 2.5.2 API-RP2A-WSD

The procedure recommended by Vesic (Winterkorn 1975) is adopted by API (2002). This method does not consider the backflow and conical effects.

\[ V_{la} = \left( c' N_c K_e + q N_q K_q + \frac{1}{2} \gamma' B N_r K_r \right) A \]  

(2.79)

Referring to equation (2.79) and Fig. 2-22, the notations are as follows.

\[ \hat{A} = 2s = BL \]

\[ L' = \left( 2s \sqrt{\frac{R+e}{R-e}} \right)^{\frac{1}{2}} \]

\[ B' = L' \frac{R-e}{\sqrt{R+e}} \]

\[ s = \frac{\pi R^2}{2} - e \sqrt{R^2 - e^2} + R^2 \sin^{-1} \left( \frac{e}{R} \right) \]

where \( A', L', B' \): effective footing area, length, width;

\( e \): eccentricity of loading;

\( K_c, K_q, K_r \): combined correction factors, taken account of loading inclination, footing shape, embedment depth, base inclination and ground slope. They are presented as follows.
\[ K_c = i_c \times s_c \times d_c \times b_c \times g_c \]
\[ K_q = i_q \times s_q \times d_q \times b_q \times g_q \]
\[ K_r = i_r \times s_r \times d_r \times b_r \times g_r \]

Loading inclination factors:

If \( \phi > 0 \),

\[ i_q = \left[ 1 - \frac{H}{Q + B L_c \cot \phi} \right]^m \]
\[ i_r = \left[ 1 - \frac{H}{Q + B L_c \cot \phi} \right]^{m+1} \]
\[ i_c = i_q - \frac{1 - i_c}{N_c \tan \phi} \]

If \( \phi = 0 \)

\[ i_c = 1 - \frac{mH}{B L_c N_c} \]

\[ m = m_L \cos^2 \theta_i + m_B \sin^2 \theta_i \]
\[ m_L = \frac{2 + \frac{L}{B}}{1 + \frac{B}{L}} \]
\[ m_B = \frac{2 + \frac{B}{L}}{1 + \frac{B}{L}} \]

where \( H, Q \), horizontal and vertical component of loading; \( c \), cohesion of clay; \( \phi \), friction angle of the soil; \( \theta_i \) is the angle between longer axis of footing and \( H \);

Footing shape factors:

\[ s_c = 1 + \left( \frac{B}{L} \right) \left( \frac{N_q}{N_c} \right) \]
Embedment depth factors:
\[ d_{q} = 1 + 2 \tan \phi \left(1 - \sin \phi\right)^2 \frac{d}{B} \]
\[ d_{r} = 1 \]
\[ d_{c} = d_{q} - \frac{1 - d_{q}}{N_{c} \tan \phi} \]

where \( d \) is embedment depth of the footing;

Base inclination factors:
\[ b_{q} = b_{r} = \left(1 - \nu \tan \phi\right)^2 \]
\[ b_{c} = b_{q} - \frac{1 - b_{q}}{N_{c} \tan \phi} \quad \phi > 0 \]
\[ b_{c} = 1 - \frac{2 \nu}{N_{c}} \quad \phi = 0 \]

where \( \nu \), base inclination angle (in radian);

Ground slope factors:
\[ g_{q} = g_{r} = \left(1 - \tan \beta\right)^2 \]
\[ g_{c} = g_{q} - \frac{1 - g_{q}}{N_{c} \tan \phi} \quad \phi > 0 \]
\[ g_{c} = 1 - \frac{2 \beta}{N_{c}} \quad \phi = 0 \]

where \( \beta \), ground inclination angle (in radian);

### 2.5.3 Houlsby & Martin (2003) ’s approach

The third method is based on the analysis done by Houlsby and Martin (2003).
They incorporated the conical shape effects, roughness of spudcan, embedment and linearly-increasing soil strength profiles. If there is no backflow, the ultimate bearing capacity is given by:

$$V_{lo} = (N_{co}c_{um} + \gamma' D_e)A + \gamma' V$$  \hspace{1cm} (2.80)

when backflow takes place, the corresponding bearing capacity is given by:

$$V_{lo} = N_{co}c_{um}A + \gamma' V$$  \hspace{1cm} (2.81)

Houlsby & Martin’s bearing capacity factors for conical footings on clay are given by

$$N_{co} = N_{coa} + \frac{\alpha_1}{\tan(\beta_c/2)} \left[ 1 + \frac{1}{6\tan(\beta_c/2)} \frac{2R\rho}{c_{um}} \right]$$

$$N_{coa} = N_{coo} \left[ 1 + \left( f_1\alpha_1 + f_2\alpha_1^2 \right) \left( 1 - f_3 \frac{h}{2R + h} \right) \right]$$

where empirical constants: $f_1=0.212$, $f_2=-0.097$, $f_3=0.53$.

$$N_{coo} = N_1 + N_2 \frac{2R\rho}{c_{um}}$$

$$N_1 = N_o \left[ 1 - f_8 \cos(\beta_e/2) \right] \left( 1 + \frac{h}{2R} \right)^{f_6}$$

$$N_2 = f_4 + f_5 \left[ \frac{1}{\tan(\beta_e/2)} \right]^{f_6} + f_7 \left( \frac{h}{2R} \right)^2$$

Empirical constants: $f_4=0.5$, $f_5=0.36$, $f_6=1.5$, $f_7=-0.4$. For smooth cones in homogeneous soil, $N_o=5.69$, $f_8=0.21$, $f_9=0.34$.

In the above formula, $\alpha_1$ is roughness factor; $\beta_c$ is equivalent cone angle of spudcan; $c_{um}$ is undrained shear strength of clay at mud line level; $\rho$ is gradient of shear strength increase of clay; $R$ is the radius of spudcan; and $h$ is embedment of spudcan (from mud line to the maximum area of the spudcan);
2.6 Summary of literature review

The complexity of jack-up superstructure and soil-structure interaction makes direct numerical analysis of the whole structure-soil interaction almost impossible. Thus, a better method to analyze soil-structure interaction is needed. This approach will capture the main feature of the response, while not causing tedious simulation problems.

A comprehensive literature review was made in this chapter. Studies conducted in past twenty years have shown that until today, a better way to efficiently incorporate spudcan fixity into structure analysis is to simulate the spudcan with structural element, taking into account the plasticity behavior of spudcan-soil interaction.

Even though SNAME (2002) provided practical guidance to simulate the fixity of spudcan, it lacks accurate theoretical basis and verification with physical modeling, compared with strain-hardening plasticity model. Van Lagen’s (1993) model is theoretical, but lacks of experimental verification. Work hardening force resultant model plays a leading role for it can easily be incorporated in structural analysis. It also gives better accuracy than the traditional pinned, spring, encastred assumptions.

As part of the plasticity model, the elastic stiffness has been thoroughly investigated by earlier researchers, including onshore and offshore conditions. Linear and nonlinear stiffness are the main objectives of these studies. The characteristics of plasticity make the non-linear stiffness less attractive. This is
because once the soil-spudcan interaction is no longer elastic, it will be governed by the flow rule and hardening law. The linear stiffness proposed in SNAME (2002), which are also conventional solutions, and coupled stiffness from Bell’s numerical study will be adopted as the basis of undrained elastic stiffness of the spudcan in the present experimental analysis. This undrained elastic stiffness will be the foundation of drained stiffness study at next stage. How the elastic rotational stiffness of spudcan varies after consolidation needs to be investigated. Yield behavior of soil is the key component to depict soil-spudcan interaction. Among the three popular yield studies, namely SNAME (2002), Langen (1993), and Martin (1994), the last one shows its soundness for it is strictly derived from experiments and gives good agreement with test results. However, its verification was done in 1g condition using small scale model, whether it is applicable in large scale spudcan or not remains questionable. One of the objectives of this study is to verify the applicability of this yield surface on larger-scale spudcan using centrifuge tests.

As indicated in Section 2.3.3.2, this model is generated from undrained condition, as would be applicable for the short-term case after installation. Field jack-up platform often operates at the same location for a relatively long time. Based on the information collected by Gan et.al (2008), two years of operation time for production wells are typical, as is reflected in Fig. 2-23. Whether this model is still applicable to the partial drained condition is unkown. If it is not applicable, is there a way to incorporate the partial drained effects in this model without
changing its main components? This question has yet been examined before and will form the studies in this thesis.

A simplified calculation of consolidation degree corresponding to 0.5, 1, 1.5, and 2 years of operation time in the field will be carried out in this study. At the same time, the degree of consolidation of kaolin clay is also estimated for comparison with that of marine clay, based on one-dimensional consolidation theory. The theoretical consolidation degrees in marine clay and kaolin clay are listed in Table 2-5 in which the marine clay parameters are taken from site investigation data on Singapore marine clay (Parsons Brinckerhoff Pte Ltd 2008).

\[
E_u = 250S_{u, ave}
\]

\[
\frac{E'}{2(1 + v')} = \frac{E_u}{2(1 + v_u)}
\]

where \(E_u, E'\) are undrained and drained elastic modulus of soil, respectively; \(v_u, v'\) are undrained and drained soil Poisson’s ratio of soil, respectively, taken as 0.49 and 0.3 respectively; and the notation of other parameters can be found in Table 2-5.

As can be seen in Table 2-5, even when the operation time is only 1 year, the degree of consolidation of marine clay and kaolin clay have reached 51.7% and 98%, respectively. As such, the undrained assumption may not be realistic and partial drained condition should be investigated. Even though a few researchers have extended the study of force-resultant model, no study has been conducted on the investigation in clay under partially drained condition, as far as the author is aware.
Table 2-5: Time factor and corresponding degree of consolidation of marine clay and KaoLin clay

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Unit</th>
<th>Marine Clay</th>
<th>KaoLin clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>k</td>
<td>m/s</td>
<td>1.00E-09</td>
<td>2.00E-08</td>
</tr>
<tr>
<td>Su,ave</td>
<td>kN/m²</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>Ea</td>
<td>kN/m²</td>
<td>5000</td>
<td>5000</td>
</tr>
<tr>
<td>m</td>
<td>m²/MN</td>
<td>6.00E-01</td>
<td>1.58E+00</td>
</tr>
<tr>
<td>Yw</td>
<td>kN/m³</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>cv</td>
<td>m²/year</td>
<td>5.26</td>
<td>40.00</td>
</tr>
<tr>
<td>d</td>
<td>m</td>
<td>10</td>
<td>10</td>
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<tr>
<td>t</td>
<td>year</td>
<td>0.5 1.5 2</td>
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<tr>
<td>Tv</td>
<td></td>
<td>0.11 0.21 0.32 0.42 0.80 1.60 2.40 3.20</td>
<td></td>
</tr>
<tr>
<td>U</td>
<td></td>
<td>36.58% 51.74% 63.37% 73.17%</td>
<td>88.74% 98.44% 99.78% 99.97%</td>
</tr>
</tbody>
</table>

Notes:
- k is permeability
- Su,ave is average undrained shear strength of the soil.
- Ea is elastic modulus of the soil
- m is coefficient of volume compressibility.
- Yw is unit weight of water.
- cv is coefficient of consolidation.
- d is soil depth.
- t is real time.
- Tv is time factor
- U is degree of consolidation corresponding to relevant time factor.
Fig. 2-1: T&R 5-5A assessment procedures of spudcan fixity (Langen 1993)

Fig. 2-2: Rotational stiffness chart (after Majer, 1958)

Fig. 2-3: Elliptical yield surface (Wiberg 1982)
Fig. 2-4: Force-displacement relation (Wiberg 1982)

Fig. 2-5: Hyperbolic moment-rotation relationship (Thinh 1984)
Fig. 2-6: Rotational stiffness vs. overturning moment (Thinh 1984)

Fig. 2-7: Footing model used in deformation analysis (Xiong 1989)
Fig. 2-8: Cases of elastic embedment for a rigid rough circular footing; Case 1, trench without backflow; case 2, footing with backflow; case 3, full sidewall contact (skirted footing) (Bell 1991)

Fig. 2-9: Typical layout of instrumentation (Nelson 2001)

Fig. 2-10: Normalized wave height (Morandi 1998)
Fig. 2-11: Comparison of dynamic fixity between measurements and T&R 5-5R (Morandi 1998)

Fig. 2-12: Lower bound of static fixity (MSL, engineering limited 2004)
Fig. 2-13: Combined loading apparatus in Oxford (Martin 1994)

Fig. 2-14: Determination of yield points through probing test (Martin 1994)
Fig. 2-15: Schematic display of tracking test (Martin 1994)

Fig. 2-16: Schematic display of looping test (Martin 1994)
Fig. 2-17: 3 leg jack up model and instrumentations in UWA (Vlahos 2001)
Fig. 2-18: Comparison of hull displacement in retrospective numerical simulations and experimental pushover results.

Figure 17 – Comparison of hull displacement in retrospective numerical simulations and experimental pushover results.

Fig. 2-18: Comparison of hull displacement in retrospective numerical simulations and experimental pushover (Cassidy 2007)
Fig. 2-19: Comparison of numerical and experimental loads on spudcans (Cassidy 2007)
Fig. 2-20: Typical comparison of experimental data and results from hyperplasticity model (Vlahos 2004)
Assume elastic spudcan response 
\[ Q_M = K_p^*, \quad Q_v = K_v^*, \quad Q_A = K_A^* \]

Perform structural assessment with pinned case inertial loadset, See Sections 5.7 & 8.

Perform foundation check at all spudcans (see 8.3.3)

Perform simplified or detailed dynamic analysis accounting for fixity?

- Simplified
  - Select initial linearised fixity
  - Run dynamic analysis using linearised fixity and determine wave force transfer functions.
  - Natural Period close to cancellation point?
    - Yes
      - Select revised linearised fixity &/or change hull mass.
    - No
      - Perform foundation checks at all spudcans (see 8.3.3)
  - Perform structural assessments
    - See Sections 5.7 & 8
      - OK
      - Not OK
        - Try more complex method
        - Yes
        - Unit NOT acceptable
        - No

- Detailed
  - Perform dynamic analysis accounting for fixity
  - (Section 6.3.4.6 Option 1)
    - Run iterative structural analysis using current inertial loadset and de-grade foundation stiffness until spudcan loads fall within foundation yield surface. Section 6.3.4.1
  - (Section 6.3.4.6 Option 2)
  - Perform structural assessments
    - See Sections 5.7 & 8
      - OK
      - Not OK
        - Try more complex method
        - Yes
        - Unit NOT acceptable
        - No

Unit acceptable

Fig. 2-21: Calculation procedure to account for foundation fixity (SNAME 2002)
Fig. 2-22: Definition of base and ground inclination of footing (Winterkorn 1975)

Fig. 2-23: Typical spudcans simulation procedure-2-year operational period (Gan 2008)
3 Design of experiments

3.1 Introduction

This chapter addresses the design and procedures of the experiments conducted in the present study. The testing plan will be elaborated. Centrifuge scale laws, the NUS centrifuge and experimental apparatus will also be introduced.

As elaborated in Section 1.2, the main objectives of this study is to assess the existing rotational stiffness theories and ultimate bearing capacity theories, verify the yield surface of force-resultant model and derive the rotational stiffness and bearing capacity variation with consolidation time. As such, elasticity, yield surface and strength increase effects of the strain-hardening model with consideration of time effects are examined. These objectives provide the basis of experimental design.

To achieve the above mentioned objectives, the spudcan-soil interaction with constant vertical displacement under combined loads in the same plane, is simulated. For single spudcan, the center of maximum area of the spudcan is regarded as the rotation center, while the top of the leg is pushed under lateral displacement control which simulates the environmental loads. At the first stage, the spudcan is penetrated to a designated depth with load control. Once the penetration is close to the designated depth, it will be switched to displacement control. The vertical load at the designated depth is recorded as the ultimate
bearing capacity of clay at that depth. Then, vertical loading will be reduced to
designated proportion of preloading under displacement control. Following that,
lateral loading will be applied with displacement control, up to pre-determined
lateral displacement. Detailed design drawings can be found in Fig. 3-1 to Fig.
3-7.

In this study, only probing tests (Martin 1994) will be carried out. Probing test is
described as follows. Firstly, the spudcan is penetrated to a certain depth where the
bearing capacity of the soil is determined as \( V_{L0} \). Second, load control will be used
to reduce the vertical force to a proportion of the bearing capacity and held at this
level. Third, lateral force will be applied under displacement control. Rotation and
lateral displacement are produced by the lateral loading. One yielding point can be
determined by this kind of test (for determination of yield point, refer to Section
5.3).

3.2 Test schedule

Four kinds of responses of spudcan under combined loads will be studied. They
are elastic behavior of spudcan, rotational stiffness, yield behavior, and bearing
capacity of normally consolidated clay under undrained and partial drained
conditions in the centrifuge.

Even though the study of spudcan under combined loads has been carried out for
two decades, few studies were related to small elastic displacement behavior. As
has been stressed by Randolph et al. (2005), “with small elastic displacement
extremely difficult to measure within a laboratory experiment, generic non-dimensional stiffness factors derived from finite element analysis combined with an appropriate choice of shear modulus is recommended.”. In fact, most of the existing solutions for combined loadings are based on FEM-derived stiffness factor. Tan (1990) put forward a new idea which links the force-resultant model with Cam clay model to explain that the yielding surface can be found through connecting sideswiping test from lower vertical unloading ratio with that from higher vertical unloading ratio when the slope ratio of virgin compression line and unloading-reloading line is large enough, say more than 100. Martin (1994) restated this idea and verified its existence. In this study, tests were taken to show the small elastic displacement response when unloading-reloading is operated during penetration.

The elastic response of rigid circular footing on the surface of homogeneous elastic half space may be obtained as reported by Poulos & Davis (1974) (see Section 2.2.1). However, this method is not strictly applicable to a circular footing with different embedment and footing roughness. Bell (1991) studied the elastic behavior of offshore shallow foundations and deduced the coupled stiffness factors for different soil Poisson’s ratio, embedment, and backflow effects using 3D FEM elastic analysis (Bell 1991). The essence of this study is summarized in Section 2.2.3.

Tests under combined loadings will be conducted in centrifuge to assess the conventional elastic theory and Bell’s elastic results. The displacements obtained
from the tests will form the inputs for the elastic matrix equation described by
Polous & Davis and Bell. The corresponding elastic combined loads from these
calculations will be compared with the test results.

As mentioned in Chapter 2, the strain-hardening force resultant model is based
on small scale spudcan experiment under 1g condition. Three tests with
penetration depth of 0.5D, 1D, 1.5D and 2D under undrained conditions will be
carried out in the centrifuge to verify the accuracy of this model. Once the
accuracy is confirmed, the partial drained tests will proceed.

The soil-spudcan interaction under partial drained condition, when consolidation
degree is less than 100%, needs to be examined if the jack up will be operated for
relatively longer time. Thus, some of the tests will be designed to experience a
designated period of consolidation. These tests will be done as follows. First, the
spudcan will be penetrated into pre-determined depth with displacement control
and then unloaded to designated service load. Immediately the lateral load will
be applied to the leg. Once the leg has been pulled back to the vertical position, it
is regarded as the starting point of soil consolidation. After a pre-determined
period, lateral load will be applied to the leg again. The process continues until
the designated stages have completely finished. Then, the spudcan will penetrate
further to the next depth. With this procedure, the spudcan reaction at scheduled
consolidation time at different depths can be obtained. Then, rotational stiffness
and bearing capacity corresponding to these responses can be determined for
further study (refer to Sections 4.3 and 6.3). At the same time, yield points will
also be determined following the methods stated in Section 5.3. Five tests will be conducted following the above-mentioned procedures. In every test, the spudcan will be penetrated into four depths: 0.5D, 1D, 1.5D and 2D. Different unloading ratio and soil consolidation time combined with different penetration depths will be tested. Unloading ratios are planned to be 0.1, 0.2, 0.3, 0.35, 0.4, 0.5, 0.6, 0.65 and 0.75. Consolidation time is designated as 0, 0.5, 1, 1.5, 2 and 3 hours in the centrifuge corresponding to prototype times 0, 0.58, 1.16, 1.74, 2.31 and 3.47 years, respectively. A total of 28 cases with various combinations will be tested in this study. The planned centrifuge tests are listed in Table. 3-1.

<table>
<thead>
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<th>Test ID</th>
<th>Test Date</th>
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<th>Unloading ratio</th>
<th>Loading time</th>
<th>Applied study</th>
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<td>1D</td>
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<td></td>
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<td>0.5</td>
<td>t=0</td>
<td></td>
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<tr>
<td></td>
<td></td>
<td>2D</td>
<td>0.4</td>
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</tr>
<tr>
<td>xj0201</td>
<td>230508</td>
<td>1D</td>
<td>0.8</td>
<td>t=0</td>
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<td></td>
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<td>1.5D</td>
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<td>xj0301</td>
<td>180708</td>
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<td>c</td>
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<td></td>
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<td>1D</td>
<td>0.5</td>
<td>t=0</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.5D</td>
<td>0.5</td>
<td>t=0</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>2D</td>
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<td>t=0</td>
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<td>xj0302</td>
<td>190708</td>
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<td>c</td>
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<td>1D</td>
<td>0.5</td>
<td>t=0</td>
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<td>1.5D</td>
<td>0.5</td>
<td>t=0</td>
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<td>2D</td>
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<td>t=0</td>
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<td>0.5</td>
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<td>c</td>
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<td>1D</td>
<td>0.5</td>
<td>t=0</td>
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<td></td>
<td></td>
<td>1.5D</td>
<td>0.5</td>
<td>t=0</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>2D</td>
<td>0.5</td>
<td>t=0</td>
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</tr>
<tr>
<td>xj0501</td>
<td>240908</td>
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<td>0.5</td>
<td>t=0,1,2 hrs</td>
<td>a,c</td>
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<tr>
<td>Test ID</td>
<td>Test Date</td>
<td>Penetration</td>
<td>Unloading ratio $V/V_{Lo}$</td>
<td>Loading time hrs</td>
<td>Applied study</td>
</tr>
<tr>
<td>---------</td>
<td>-----------</td>
<td>-------------</td>
<td>-----------------------------</td>
<td>------------------</td>
<td>---------------</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1D</td>
<td>0.5</td>
<td>t=0,1,2 hrs</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.5D</td>
<td>0.5</td>
<td>t=0,2 hrs</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>2D</td>
<td>0.5</td>
<td>t=0,2 hrs</td>
<td></td>
</tr>
<tr>
<td>xj0601</td>
<td>021008</td>
<td>0.5D</td>
<td>0.35</td>
<td>t=0,2 hrs</td>
<td>a,c</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1D</td>
<td>0.6</td>
<td>t=0,2 hrs</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.5D</td>
<td>0.75</td>
<td>t=0,1,2 hrs</td>
<td></td>
</tr>
<tr>
<td>xj0602</td>
<td>041008</td>
<td>0.5D</td>
<td>0.2</td>
<td>t=0,2 hrs</td>
<td>a,c</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1D</td>
<td>0.3</td>
<td>t=0,2 hrs</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.5D</td>
<td>0.1</td>
<td>t=0,2 hrs</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>2D</td>
<td>0.4</td>
<td>t=0,2 hrs</td>
<td></td>
</tr>
<tr>
<td>xj0603</td>
<td>061008</td>
<td>0.5D</td>
<td>0.75</td>
<td>t=0,1 hrs</td>
<td>a,c</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1D</td>
<td>0.65</td>
<td>t=0,1,3 hrs</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.5D</td>
<td>0.2</td>
<td>t=0,1,3 hrs</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>2D</td>
<td>0.4</td>
<td>t=0,1,3 hrs</td>
<td></td>
</tr>
<tr>
<td>xj0701</td>
<td>221008</td>
<td>0.5D</td>
<td>0.2</td>
<td>t=0,0.5,1,1.5 hrs</td>
<td>a,c</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1D</td>
<td>0.4</td>
<td>t=0.51,5 hrs</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.5D</td>
<td>0.6</td>
<td>t=0.0,51,5 hrs</td>
<td></td>
</tr>
</tbody>
</table>

Notes:
1. a represents rotational stiffness study; 
   b represents yield surface study; 
   c represents bearing capacity study; 
2. D is diameter of spudcan; 
   $V/V_{Lo}$ is the unloading ratio 
   V is the unloaded value and 
   $V_{Lo}$ is the pre-loaded value;

### 3.3 Jack-up physical model

In a joint industry study, Noble Denton Europe conducted a series of field investigations in the North Sea. Nataraja (2004) and Nelson et.al (2001) published their measurement reports. It is found that the GSF Magellan was one of the most investigated jack-ups. Its operation locations, corresponding environmental conditions, and some of the soil profiles were reported in detail. Thus, the field model Magellan was selected as the model to simulate.
Owing to limitation in the centrifuge dimension, the prototype is scaled to one third of the original dimension. The centrifuge model is created with existing scaling theory as shown in Table. 3-2. The data indicating centrifuge, prototype, and field are presented in Table. 3-3.

Table. 3-2 Scaling relations (Leung 1991)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Prototype</th>
<th>Centrifuge model at ( Ng )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear dimension</td>
<td>1</td>
<td>( 1/N )</td>
</tr>
<tr>
<td>Area</td>
<td>1</td>
<td>( 1/N^2 )</td>
</tr>
<tr>
<td>Volume</td>
<td>1</td>
<td>( 1/N^3 )</td>
</tr>
<tr>
<td>Density</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Mass</td>
<td>1</td>
<td>( 1/N^3 )</td>
</tr>
<tr>
<td>Acceleration</td>
<td>1</td>
<td>( N )</td>
</tr>
<tr>
<td>Velocity</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Displacement</td>
<td>1</td>
<td>( 1/N )</td>
</tr>
<tr>
<td>Strain</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Energy density</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Energy</td>
<td>1</td>
<td>( 1/N^3 )</td>
</tr>
<tr>
<td>Stress</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Force</td>
<td>1</td>
<td>( 1/N^2 )</td>
</tr>
<tr>
<td>Time(creep)</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Time(dynamics)</td>
<td>1</td>
<td>( 1/N )</td>
</tr>
<tr>
<td>Time(consolidation)</td>
<td>1</td>
<td>( 1/N^2 )</td>
</tr>
</tbody>
</table>

Table. 3-3 Jack-up model description

<table>
<thead>
<tr>
<th>Items</th>
<th>Model</th>
<th>Prototype</th>
<th>Field</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length overall(m)</td>
<td></td>
<td></td>
<td>69.5</td>
</tr>
<tr>
<td>Width Overall(m)</td>
<td></td>
<td></td>
<td>66.6</td>
</tr>
<tr>
<td>Depth of Hull(m)</td>
<td></td>
<td></td>
<td>9.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>--------------------------------</td>
<td>--------</td>
<td>-------</td>
<td>------</td>
</tr>
<tr>
<td>Total Length of legs (m)</td>
<td></td>
<td></td>
<td>148.8</td>
</tr>
<tr>
<td>Water depth (m)</td>
<td>0.3</td>
<td>30</td>
<td>92.7</td>
</tr>
<tr>
<td>Ctr of F leg to ctr of aft leg (m)</td>
<td>0.13</td>
<td>13</td>
<td>45.7</td>
</tr>
<tr>
<td>Ctr to ctr of aft leg (m)</td>
<td>0.15</td>
<td>15</td>
<td>47.5</td>
</tr>
<tr>
<td>Spudcan diameter (m)</td>
<td>0.06</td>
<td>6</td>
<td>18</td>
</tr>
<tr>
<td>EI of leg (Nm²/rad)</td>
<td>268 mm²</td>
<td>2.68E10</td>
<td>2.17E12</td>
</tr>
<tr>
<td>Cross section area (m²)</td>
<td>13.3mm²</td>
<td>0.133</td>
<td>1.2</td>
</tr>
<tr>
<td>Self weight (kN)</td>
<td>1.522</td>
<td>15220</td>
<td>136980</td>
</tr>
<tr>
<td>Penetration (m)</td>
<td></td>
<td></td>
<td>2.4, 4.6</td>
</tr>
</tbody>
</table>

### 3.4 Experimental apparatus

#### 3.4.1 Centrifuge and control system

The experiments were carried out on the National University of Singapore centrifuge. The 2m arm centrifuge has a capacity of 40 g-tonnes; that is the maximum payload is 400kg at 100g. Load is applied through existing laboratory Deublin hydraulic units which provide a maximum of 1000psi pressure. Two branches are connected with vertical and lateral cylinders respectively. The feedback consists of potentiometer, laser sensor, load cell, amplifier, control system, servo. The process is as follows. The potentiometer transmits signal to computer through amplifier. The computer then sends command to adjust the servo valve after data comparison. This process is close loop and will be terminated until the load coincides with the required displacement (load).

#### 3.4.2 Instrumentation apparatus

The spudcan penetration depth is measured by a 300 mm travel potentiometer.
resting on the stainless steel girder and horizontal displacement is measured and
controlled by two 200-mm laser sensors which are also the tools to capture the
spudcan rotation. Two loadcells are used to measure the loads. Among them,
Interface WMCa-53 1k is selected to measure the vertical load and Interface
SML-51 500 is used to record the lateral load. To ensure the accuracy and facilitate
verification, 3-level full bridge axial strain gauges are installed along the shaft of
the jack-up leg. 3-level half bridge strain gauges are used to measure the bending
moment of the leg, as indicated in Fig. 3-2. The layout of strain gauges for these
two functions is illustrated in Fig. 3-8. Meanwhile, two laser sensors
Micro-Epsilon ILD1300-200 are installed at the side of the frame to measure the
displacement of the leg top and bottom. The specifications of the above mentioned
instrumentation apparatus are listed in Table. 3-4.

Table. 3-4 Summary of instrumentation apparatus in centrifuge test

<table>
<thead>
<tr>
<th>measure_range</th>
<th>laser_top</th>
<th>laser_bot</th>
<th>LVDT_V</th>
<th>LVDT_tbar</th>
<th>Lc_500</th>
<th>Lc_1k</th>
<th>Dw</th>
</tr>
</thead>
<tbody>
<tr>
<td>unit</td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
<td>lbf</td>
<td>lbf</td>
<td>inch</td>
</tr>
<tr>
<td>output_range</td>
<td>5</td>
<td>5</td>
<td>10</td>
<td>10</td>
<td>18.9185</td>
<td>71.616</td>
<td>10</td>
</tr>
<tr>
<td>unit</td>
<td>V</td>
<td>V</td>
<td>V</td>
<td>V</td>
<td>mV</td>
<td>mV</td>
<td>V</td>
</tr>
<tr>
<td>factor</td>
<td>0.02004</td>
<td>0.02125</td>
<td>0.03342</td>
<td>0.03342</td>
<td>0.00413</td>
<td>-0.0161</td>
<td>0.01467</td>
</tr>
<tr>
<td>unit</td>
<td>v/mm</td>
<td>v/mm</td>
<td>v/mm</td>
<td>v/mm</td>
<td>mv/N/10V</td>
<td>mv/N/10V</td>
<td>v/mm</td>
</tr>
<tr>
<td>Strain gauge</td>
<td>Axial-1</td>
<td>Axial-2</td>
<td>Axial-3</td>
<td>Bm-4</td>
<td>Bm-5</td>
<td>Bm-6</td>
<td></td>
</tr>
<tr>
<td>Unit</td>
<td>N/με</td>
<td>N/με</td>
<td>N/με</td>
<td>N.mm/με</td>
<td>N.mm/με</td>
<td>N.mm/με</td>
<td></td>
</tr>
</tbody>
</table>
| Notes:        | laser denotes laser sensor
|               | Dw denotes drawwire |
|               | Lc denotes loadcell |
3.4.3 Test setup

Lateral movement of the spudcan-leg is measured by two laser sensors. Vertical distance is measured by 300 mm travel potentiometers. The lateral and vertical loading are measured by 300 lbf and 1000 lbf load cell. The readings of these instruments all are displayed in voltage on the software Dasylab. The readings of strain gauges, including axial force and bending moment gauges, are collected by an independent strain meter through which the data are transferred to the PC in the control room. In this study, the strain gauges measuring axial force will be installed with full bridge circuit and the half bridge is adopted for bending moments measurements (Kyowa sensor system 2008). These two electrical layouts are given in Fig. 3-8. A special software named as “static instrument” is used to store the strain data into PC. The calibration factors of these instrumentation apparatus are appended in Table. 3-4. Data collection frequency of Dasylab is set up to 100 Hz and the average No is 50. This leads to 2 Hz frequency of data storage. Data collection frequency of strain meter is setup to 1Hz. The whole setup in the centrifuge is presented in Fig. 3-9.

The clay used in this study is Malaysia kaolin clay. Its properties have been investigated by many researchers. Some of the main properties are listed in Table. 3-5 (Goh 2003). The standard procedures in geotechnical laboratory of National University of Singapore are followed to prepare the kaolin clay. Many
researchers have mentioned these procedures (Goh 2003).

Table 3-5: Properties of Malaysia kaolin clay (Goh 2003)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
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<tr>
<td>Liquid limit ($w_l$)</td>
<td>%</td>
<td>80</td>
</tr>
<tr>
<td>Plastic limit ($w_p$)</td>
<td>%</td>
<td>35</td>
</tr>
<tr>
<td>Specific gravity, $G_s$</td>
<td></td>
<td>2.6</td>
</tr>
<tr>
<td>Consolidation coefficient (at 100Kpa), $c_v$</td>
<td>m²/year</td>
<td>40</td>
</tr>
<tr>
<td>Permeability on NC clay (at 100Kpa), $k$</td>
<td>m/sec</td>
<td>2.0E-08</td>
</tr>
<tr>
<td>Angle of internal friction, $\phi'$</td>
<td></td>
<td>23</td>
</tr>
<tr>
<td>Modified Cam-Clay parameters:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$M$</td>
<td></td>
<td>0.9</td>
</tr>
<tr>
<td>$\lambda$</td>
<td></td>
<td>0.244</td>
</tr>
<tr>
<td>$\kappa$</td>
<td></td>
<td>0.053</td>
</tr>
<tr>
<td>$N$</td>
<td></td>
<td>3.35</td>
</tr>
</tbody>
</table>

3.5 Analysis strategies

The data collected from Dasylab will be averaged for every second in order to ease the processing of combined data. This is done by the VBA subroutine extract_averageddata. However, the data collected in Dasylab suite and strain meter are not simultaneous due to the frequency instability of the strain meter. This problem is solved in subroutine Compare_delete.

Under 100g, the readings of all apparatus before spudcan penetration are taken as the initial readings. Subsequent response of the system is obtained through subtraction of this initial reading when penetration occurs or lateral loading is applied. The calibration factors listed in Table 3-4 will be taken into account to transform the voltage readings into meaningful data respectively. The above mentioned process is implemented in Excel sheet 2.

The data required in next step will be extracted into Excel sheet 7 where they will
be processed in terms of the following procedures. The sign convention of the spudcan is shown in Fig. 3-10 where the right, downward, clockwise are defined as positive for lateral force, vertical force and bending moment respectively. The jack-up leg and spudcan are simulated as rigid body. When the leg is pushed, the lateral force will be calculated from load cell SML-51-300 and vertical force is obtained through WMCa-53-1k load cell, as the bending moment is measured by half-bridge strain gauges. When the rotation of centrifuge has been stable in 100g and the penetration has yet been conducted, the readings at this stage are regarded as the initial reference. Once the laser sensors capture displacement variation, pushing (or pulling) takes place. There are four cases displaying the leg-spudcan response under combined loads. The rotation angle of spudcan is calculated from the reading difference of top laser and bottom laser. The horizontal displacement of the spudcan will be obtained through the relationship of top/bottom laser reading and reference point variation. Vertical force defined in sign convention will be the cosine part of the measured force, as will be represented as a component force. For the convenience of comparison, all the forces and displacements are generalized as prototype according to centrifuge scale rule. The calculation is carried out through subroutine Analyze_prototype. This processing is detailed in flowchart Fig. 3-11.

3.6 Concluding remarks

In this chapter, the design of the experiments is presented. The test schedule to
achieve the study objectives is briefly introduced. The tests can be generalized as elastic and plastic response of spudcan under combined loads. Several tests are designed to assess the existing elastic theories of the spudcan under undrained conditions. Three tests are designed to verify the yield surface of the strain-hardening force resultant model with centrifuge modeling under undrained conditions. A total 28 cases of different combinations of penetration depth, unloading ratio and consolidation time were proposed to investigate the partial drained effects on rotational stiffness and ultimate bearing capacity of the spudcan. Based on these tests, the existing bearing capacity theories will be assessed and the variation of the elastic stiffness and bearing capacity of spudcan under partial drained conditions will be investigated.
Tub and Loading equipment

Fig. 3-1: Apparatus design-1
Fig. 3-2: Apparatus design-2
Joint Hinge Connection between Spudcan and Cylinders

Fig. 3-3: Apparatus design-3
Fig. 3-6: Apparatus Design-6
Fig. 3-8: Half bridge and full bridge illustrations for the measurement of bending moment and axial force respectively (Kyowa sensor system 2008)

Fig. 3-9: Setup in centrifuge
Fig. 3-10: Sign convention adopted by this study

Read constant parameters
Dis_laser: distance between top laser and bottom laser
Toplaser_spud: distance between top laser and reference point of spudcan;
Botlaser_spud: distance between bottom laser and reference point of spudcan;

Acquire initial readings of all variables;

Calculate theta, u, w, V, H, M in prototype for every increment of displacement;

Write the data into another sheet;

End of the data?

No

Yes

End sub

Fig. 3-11: Flowchart for the processing of centrifuge data
4 Rotational stiffness of spudcan foundation

4.1 Introduction

The rotational stiffness of the spudcan under undrained condition and its variation under partially drained condition will be investigated in this chapter. Tests were done to obtain the parameters of the soil so that they can be applied to the experimental study at a latter stage. An appropriate method will be determined to obtain the initial rotational stiffness of the spudcan immediately after it has penetrated to a certain depth in the centrifuge tests. This method will provide the basis to study the rotational stiffness variation with soil consolidation time. Then, centrifuge tests under partially drained condition will be conducted and analyzed. An empirical relationship between the rotational stiffness of the spudcan after soil consolidation and relevant variables, such as consolidation time, unloading ratio and initial rotational stiffness, will be generalized.

4.2 Stiffness and Poisson’s ratio used in this study

Existing guidelines (SNAME 2002) provide a semi-empirical relationship of the soil shear modulus and strength as follows:

\[
\frac{G_u}{C_u} = \begin{cases} 
50 & \text{OCR}>10 \\
100 & 4<\text{OCR}<10 \\
200 & \text{OCR}<4 
\end{cases} 
\]  

(4.1)

Undrained soil Poisson’s ratio, \(\nu_u=0.5\)
Effective shear modulus is given by: $G' = G_u$

The “at rest” soil Poisson’s ratio is used as the effective Poisson’s ratio in this study:

\[
ν_o = \frac{κ_o}{1 + κ_o} = \frac{1 - \sin \phi}{2 - \sin \phi}
\]  

(4.2)

where the at rest earth pressure $κ_o$ is determined by Jaky’s law.

In the latter analysis of experimental data, the shear modulus of the soil will be obtained using the above formula.

### 4.3 Determination of initial rotational stiffness

The main objective of these tests is to find a suitable method to determine the initial rotational stiffness of spudcan for further study under partially drained conditions. The initial rotational stiffness used in this study is defined as the secant rotational stiffness of the spudcan before yielding takes place. Two well-known theories were adopted in this analysis, Bell (1991)’s FEM study and conventional theory suggested by SNAME (2002).

Undrained shear strength, $C_u$, will be adopted from the T-bar test at the designated depth where the spudcan reaction will be analyzed. The corresponding shear modulus of the clay will be determined with the method recommended in Section 4.2 where OCR<4.

Based on the maximum curvature principle (see Section 5.3), the yield points will be determined. The corresponding $M/R^2 \theta$ for every yield point can be readily calculated from the processed test data. This value is regarded as the
rotational stiffness of the spudcan, where $M$ is the yield moment, $R$ is the radius of spudcan, $\theta$ is rotation angle of the spudcan at yield (in radian). All the experimental rotational stiffness values, which are determined using this method, are given in Table. 4-1. The main objective here is to find the rotational stiffness variation under partial drained condition. As the constant $R$ would not affect the definition of stiffness, all the tests are conducted using the same spudcan model.

The following tests are used to assess the elastic rotational stiffness: xj0201_1D, xj0201_2D, xj0402_0.5D, xj0402_1D, xj0402_1.5D and xj0402_2D. The last two characters represent the penetration depth, for example, 1D is the depth at one spudcan diameter from ground level. The first two samples are tested to failure, while the remaining four tests are done in 1 cycle with relative smaller rotation angle. For the tests shown in Fig. 4-1 and Fig. 4-2, only the rotational stiffness resulting from Bell’s FEM (equation(2.40)) is utilized to compare the response of spudcan under combined loads. The combined loads obtained from centrifuge tests will be taken as the input of the right side of the equation. Thus, the displacements corresponding to every force combination will be obtained. The same principle is applied to the SNAME (2002) formulations (equation(2.32)) for the subsequent four tests. Then, the moment and rotation angle from the tests and the above mentioned analysis are plotted on the same plane for comparison. As can be seen from Fig. 4-3 and Fig. 4-4, these results reveal the same tendency that the analysis based on Bell’s theory always predicts much stiffer soil behavior than the one using SNAME (2002). The prediction from elastic theories is close to the
measured value only when the rotation angle is less than 0.3°. After that rotation, the moment resistance of soil is shown to be non-linear. xj0402-1D will be taken as an example. As shown in Fig. 4-3, the largest rotation angle predicted by elastic theory at the maximum moment during test is about 0.42°; while with the same moment, the measured rotation is 4.96°. It is self evident that the conventional elastic theory will produce unsafe displacement in site assessment. The rest of the tests also show the same trend as above mentioned.

4.4 Rotational stiffness variation due to consolidation

As mentioned in chapter 2, time effects on soil-spudcan interaction are important when the jack-up sits on the seabed for a relatively long time. To investigate the time effects on rotational stiffness of spudcan, a series of tests were done in the centrifuge. First, the spudcan will be penetrated to a pre-determined depth with displacement control and then unloaded to designated service load. Immediately the lateral load will be applied to the leg. Once the leg has been pulled back to the vertical position, it is regarded as the starting point of consolidation. After a pre-determined period of consolidation, lateral load will be applied to the leg again. The process continues until the tests designed for this depth have been completed. Then, the spudcan will be penetrated to the next depth. With this procedure, the spudcan reaction at scheduled soil consolidation times at different depths can be obtained. The consolidation time of the clay in the centrifuge is transformed into real time according to centrifuge scaling laws (Leung 1991).
The tests including five batches (total 18 different depths) are demonstrated in Table. 4-1 in which rotational stiffness $k_{ro}$ at the time when the spudcan just penetrated to the desired depth will be taken as the base for comparison. At the subsequent time represented by $t$, the rotational stiffness $k_{rt}$ is normalized by $k_{ro}$. That is how the stiffness multiplier is obtained. It is established from the analysis presented in the last section that the the initial rotational stiffness $k_{ro}$ have been normalized by soil shear modulus and spudcan radius. Thus, the stiffness multiplier will mainly be related to the unloading ratio $n$ (defined as the ratio of working load and ultimate bearing capacity of the spudcan at a certain depth) and consolidation time of the clay. That is,

$$k_{rt} \sim f(k_{ro}, n,t)$$

The data have been processed as follows. First, several unloading ratios $n$ are chosen to be the basis of grouping, such that $n=0.2, 0.4, 0.48, 0.6, 0.65, 0.75$. For each unloading ratio, the rotational stiffness is a function of corresponding soil consolidation time. It is estimated that excess pore pressure produced by centrifuge force could be dissipated fully within 6 hours in kaolin clay. The static rotational stiffness will be close to an asymptote value after a period of standing. In addition, from conventional consolidation theory the pore pressure dissipation is faster at the beginning of the process than in the later stage. It is reasonable to fit the relationship of rotational stiffness and time with a hyperbolic line. A new parameter is defined here.

$$\beta = \frac{k_{rt}}{k_{ro}}$$

(4.3)
where $\beta$ is rotational stiffness multiplier, $k_{rt}$ is rotational stiffness of clay at time $t$, $k_{ro}$ is initial rotational stiffness of clay at $t=0$.

The fact that all the normalized initial rotational stiffness start from 1 makes it reasonable to subtract 1 from all rotational stiffness multipliers. Thus, the fitted hyperbolic line can be presented as,

$$\frac{t}{\beta - 1} = a + bt \quad (4.4)$$

where $a$, $b$ are fitted coefficients.

Test results and sorted data are shown in Table. 4-2.

For every unloading ratio $n$, linear fitting was conducted to obtain $a$ and $b$. The fitted results are presented in Table. 4-3. Linear fitted lines corresponding to every unloading ratio are shown in Fig. 4-7, Fig. 4-9, Fig. 4-11, Fig. 4-13, Fig. 4-15 and Fig. 4-17. The relationship of $\beta$ and consolidation time are correspondingly displayed in Fig. 4-8, Fig. 4-10, Fig. 4-12, Fig. 4-14, Fig. 4-16 and Fig. 4-18.

As discussed above, another important factor, the unloading ratio, may affect the variation of rotational stiffness with time. To investigate this hypothesis, some trials were carried out to look into the relationship. It is found that coefficient $a$ versus $n$ can be fitted with a parabolic curve and presented as:

$$a = 0.31 - 2.314n + 6.018n^2 \quad (4.5)$$

The fitted line with equation (4.5) and test data are presented in Fig. 4-19. The coefficient of determination of this fitting, $r^2$, is 0.82, which shows that the fitting is good.
Similar principle was applied to fit $b$ versus $n$ which could be displayed as a constant,

$$b = 0.6$$  \hspace{1cm} (4.6)

The fitted line with equation (4.6) and points fitted with test data are shown in Fig. 4-20.

**4.5 Summary**

A study was done to assess the existing elastic rotational stiffness theories. Bell’s method can be regarded as a good method to determine the initial stiffness of the spudcan due to its comprehensive consideration (embedment depth, soil Poisson’s ratio, backflow effects, shape effects are taken into consideration) and robust numerical basis, compared with the conventional method for surface footings.

The rotational stiffness variation after a certain period of soil consolidation is experimentally generalized. It is concluded that the rotational stiffness is a function of initial stiffness, time and unloading ratio. This rotational stiffness can be readily incorporated into the strain-hardening model, with the consideration of time effects.
Table 4-1: Centrifuge test results presenting rotational stiffness variation with time and unloading ratio

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Table. 4-2: Processed rotational stiffness variation according to time and unloading ratio

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Table. 4-3: a, b coefficients with unloading ratio

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Fig. 4-1: M-theta plot of measurement and coupled elastic stiffness theory for test xj0201-230508 at 1D penetration

Fig. 4-2: M-theta plot of measurement and coupled elastic analysis for test xj0201-230508 at 2D penetration
Fig. 4-3: M.vs.theta response of test xj0402-200808 at 0.5D penetration for the case of 1 cycle

Fig. 4-4: M.vs.theta plot for test xj0402-200808 at 1D penetration for 1 cycle case
Fig. 4-5: M.vs.theta plot for test xj0402-200808 at 1.5D penetration for the case of 1 cycle

Fig. 4-6: M.vs.theta plot for test xj0402-200808 at 2D penetration for the case of 1 cycle
Fig. 4-7: Linear fitting of $t/(\beta - 1)$ with $t$ for $n=0.2$

\[ t / (\beta - 1) = 0.662 + 0.284t \]

Fig. 4-8: Rotational stiffness variation with time for unloading ratio $n=0.2$
Fig. 4-9: Linear fitting of $\frac{t}{(\beta -1)}$ with $t$ for $n=0.4$

Fig. 4-10: Rotational stiffness variation with time for unloading ratio $n=0.4$

\[ \frac{t}{(\beta -1)} = 0.2365 + 1.093t \]
Fig. 4-11: Linear fitting of $t/(\beta - 1)$ with $t$ for $n=0.48$

$$t/(\beta - 1) = 0.522 + 0.302t$$

Fig. 4-12: Rotational stiffness variation with time for unloading ratio $n=0.48$
Fig. 4-13: Linear fitting of $\frac{t}{(\beta - 1)}$ with $t$ for $n=0.6$

\[
\frac{t}{(\beta - 1)} = 0.272 + 0.715t
\]

Fig. 4-14: Rotational stiffness variation with time for unloading ratio $n=0.6$
Fig. 4-15: Linear fitting of $t/(\beta - 1)$ with $t$ for $n=0.65$

$$t/(\beta - 1) = 1.854 + 0.724t$$

Fig. 4-16: Rotational stiffness variation with time for unloading ratio $n=0.65$
Fig. 4-17: Linear fitting of $t/(\beta - 1)$ with $t$ for $n=0.75$

$t/(\beta - 1) = 2.057 + 0.17t$

Fig. 4-18: Rotational stiffness variation with time for unloading ratio $n=0.75$
Fig. 4-19: Parabolic fitting of coefficient a with unloading ratio n

Fig. 4-20: Elliptical fitting of coefficient b with unloading ratio n
5 Verification of yield surface of strain-hardening force resultant model under undrained condition

5.1 Introduction

The strain-hardening model developed by Houlsby and Martin (1994) has been discussed in Chapter 2. The derivation of model and verification with one leg or three legs spudcan were all conducted in 1g using small spudcans. It is therefore necessary to further verify this model at large scale. A series of tests simulating the stress state of large scale soil-spudcan interaction were performed in NUS centrifuge.

The idea of tracking tests (Martin 1994) which play a significant role in the development of strain-hardening force resultant model was developed from the hypothesis that the yield behavior of spudcan foundation may be similar to the yielding of soil simulated by the modified Cam Clay model. Tan (1990) verified this with circular footing on sand in the centrifuge. Martin’s tests (Martin 1994) on spudcan under 1g condition strongly supported this hypothesis.

5.2 Similarity of constitutive model and force-resultant model

How the ratio, $\lambda/\kappa$, in modified Cam Clay model affects the yield surface in
undrained triaxial tests is clearly stated by Martin (1994). In Fig. 5-1, two cases with the same parameters of modified Cam Clay ($\lambda =0.25$, $M=0.9$, $N=3.5$, $G' =1000 \text{kPa}$) except $\kappa =0.05$ in case (a) and $\kappa =0.005$ in case (b) are compared. The first case with $\lambda / \kappa =5$ shows that the soil is isotropically compressed to point A. After that, two operations will be conducted under undrained triaxial test. One is the soil will be compressed to critical state line along the path A-B. Another is unloaded to the point X where the hydrostatic pressure $p' =0$, then compressed to the critical state line along path X-Y-Z under undrained condition. The second case with $\lambda / \kappa =50$ shows the similar triaxial tests progress under undrained condition. However, these two cases displayed different yielding responses at the end of undrained compression from the low unloading ratio points (X and X’) to the high unloading ratio points (A and A’). As shown in Fig. 5-1, the yield surfaces presented by A’-B’ and X’-Y’-Z’ can simulate the yield surface characterized by point A much better than those presented by A-B and X-Y-Z due to the smaller difference in the hydrostatic stress, $\delta p'$, between point B’ and Z’. This is explained by the fact that in undrained triaxial tests, the volumetric strain, namely the sum of elastic volumetric strain and plastic volumetric strain, will be zero, $\delta e_v^e + \delta e_v^p =0$. Thus, if the elastic modulus of unloading-reloading, $\kappa$, becomes smaller, the elastic volumetric strain will be smaller correspondingly for a given $\delta p'$. This implies a smaller plastic volumetric strain, $\delta e_v^p$.

The centrifuge tests done by Tan (1990) with flat circular footing on sand
confirmed the possibility of plotting the yield locus in V-H plane with two tests. The results of his tests are shown in Fig. 5-2. The footing was penetrated to point B, then unloaded to point C and reloaded to point D, see Fig. 5-2. After that, the footing was horizontally pushed along the path D-E-F, while the vertical displacement was held constant, defined as “sideswipe test” by Tan (1990), and “Tracking test” by Martin (1994). Another test was penetrated to point D and unloaded to point G where the unloading ratio is very low (Fig. 5-2). Horizontal “sideswipe test” was applied to this test along the path G-H. It is shown that the yield loci of these two tests, D-E-F and G-H, can almost describe the yield locus governed by point D, provided that the ratio of loading-unloading gradient, \( \frac{F_{\text{vir}}}{F_{\text{ur}}} \) (\( F_{\text{vir}} \) is the gradient of virgin penetration line, \( F_{\text{ur}} \), is the gradient of unloading-reloading line, \( F_{\text{vir}}/F_{\text{ur}} \) is defined as flexibility ratio), is large enough (Tan’s test is in the order of 50~100). Tan compared the yield locus obtained from sideswipe tests with his limited results from probing tests and confirmed that sideswipe tests indeed could depict the yield locus easily. Dean et al (1992) extended the application of this method to other planes, such as V-M and H-M. Martin (1994) derived the strain-hardening force resultant model in clay mainly from the “tracking tests” with a high ratio of loading-unloading gradient. As such, this important basis will be investigated in kaolin clay in this study.

In the present study, two unloading-reloading cases are extracted from test xj0201-230508 (Table. 3-1) for loading-unloading analysis. One is at 5.3m depth, and another at 9.8m. An overview of the unloading-reloading response at these
two depths is plotted in Fig. 5-3. To make this response clear, two independent analyses of unloading-reloading behavior were carried out at 5.3m and 9.8m penetration depth, respectively. For the case of 5.3m penetration, the detailed unloading and reloading data in mm versus kN is plotted in Fig. 5-4 (where $V_c$ is the vertical loading). It can be seen that the maximum displacement is 36.65mm, corresponding to vertically unloading to 2760kN from the virgin compression loading 5180kN. The linear fitted line of the unloading-reloading data is also shown in Fig. 5-4. To compare with the test results, existing theories would be utilized. The more accurate results accounting for the soil Poisson’s ratio, embedment, and backflow obtained using 3D FEM (vertical stiffness of spudcan in equation (2.39)) can be regarded as a better estimation of the stiffness of the foundation. For this comparison work, the SNAME (2002) recommended rigidity index is applied to the normally consolidated soil, that is,

$$\frac{G}{C_u} = 200$$

(5.1)

where $G$ is undrained shear modulus of soil, $C_u$ is undrained shear strength of clay.

The $C_u$ value at the depth of spudcan reference point will be taken into consideration at every stage.

Undrained soil Poisson’s ratio, $\nu_u=0.49$

In order to simulate the penetration response more accurately, backflow effects are taken into account following Meyerhof(1972)’s suggestion. In his slurry trench study, Meyerhof provided the relationship of stability number $N$ and ratio of depth
and width of trench, D/B. This relationship can be well presented by a cubic line through non-linear polynomial fitting.

\[ N = 0.00802\left(\frac{D}{B}\right)^3 - 0.17537\left(\frac{D}{B}\right)^2 + 1.30655\left(\frac{D}{B}\right) + 4.32011 \]  \hspace{1cm} (5.2)

The fitted line and Meyerhof’s (1972) original data are plotted in Fig. 5-5. Several researchers verified that the stability number provided by Meyerhof is conservative in practice (Menzies 2008).

The following equation shows backflow occurs in the field (SNAME 2002):

\[ D > \frac{Nc_{ur}}{\gamma} \]  \hspace{1cm} (5.3)

Referring to this criterion, the soil vertical stiffness is calculated from Bell’s study.

The linear interpolation is applied to the soil Poisson’s ratio and embedment ratio if they are not in this table. For the convenience of comparison, flexibility which reflects the ratio of displacement (in m) and loading (in kN) is chosen as the main parameter. As shown in Fig. 5-6, the unloading-reloading gradient at 5.3m depth is 1.569E-5, while the gradient of the virgin penetration line at the same depth is 1.36E-3. \( \frac{F_{vir}}{F_{ur}} = 1.36E-3/1.569E-5 = 86.68 \)

where \( F_{vir} \) is the gradient of virgin penetration line, \( F_{ur} \) is the gradient of unloading-reloading line, \( F_{vir}/F_{ur} \) is defined as flexibility ratio.

Comparing to laboratory test results, such as the kaolin clay used \( \lambda / \kappa = 0.244/0.053 = 4.6 \) (Goh 2003) in the present study, the flexibility ratio in force-resultant model is 20 times larger. This verifies that the yielding surface can be reasonably constructed from high ratio and low vertical loading conditions in
swiping tests.

The unloading-reloading flexibility ratio is shown in Table 5-1 comprising the results from Bell’s 3D FEM (equation (2.39)) and the results from SNAME (2002) (equation (2.30)). It is noted that the measured unloading-reloading gradient is much closer to the value from SNAME’s recommendation which is based on a rigid surface footing, although the same shear modulus $G_u$, soil Poisson’s ratio $\nu_u$, and spudcan radius $R$ value are adopted for both methods. The difference is that Bell’s analysis takes account of embedment and backflow effects, while SNAME (2002) does not account for them. It is interesting to find such behavior that contradicts with normal judgment. The prediction by Bell’s method is 37% stiffer than the measured soil stiffness.

Table 5-1: Flexibility comparison of unloading reloading response at two depths

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Measured</th>
<th>Equation (2.39) following Bell (1991)</th>
<th>Equation (2.30) following SNAME (2002)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.3</td>
<td>1.57E-05</td>
<td>1.147E-05</td>
<td>1.519E-05</td>
</tr>
<tr>
<td>9.8</td>
<td>9.50E-06</td>
<td>5.807E-06</td>
<td>8.385E-06</td>
</tr>
</tbody>
</table>

The same principle is applied to the unloading reloading case at 9.8m penetration. As shown in Fig. 5-8 and Fig. 5-5, the UR line gradient is 9.5E-6 which indicates that the soil stiffness is 1.65 times stiffer than the one at 5.3m. The slope of virgin penetration line is 6.93E-4, representing a flexibility ratio $F_{vir}/F_{ur}$=72.97 which is slightly smaller than the value at 5.3m, but this high ratio also proves the applicability of tracking test. Once again, the measured flexibility shows its
approximation to the prediction from SNAME’s surface footing and a little farther from the FEM study. These two uniform discrepancies may be resulted from the following possibilities. First, Bell’s model does not account for the conical shape effect and linearly increasing soil profile. Second, perfectly rough footing is assumed, and this method increase the soil constraint on the footing. Third, the zero thickness interface element may also be an attribution to the discrepancy as it is not able to capture the status of loss of contact under combined loading. Finally, the determination of shear modulus may lead to some difference between different methods.

In this study, the large flexibility ratio of loading-unloading-reloading behavior is verified by centrifuge tests. As elaborated by Martin (1994), this large flexibility ratio of loading-unloading-reloading behavior of spudcan is the prerequisite to deduce the force-resultant theory from tracking tests. Thus, the validity of existing force-resultant model is partially verified.

5.3 Determination of yield points

In this study, the yield points are determined using the greatest curvature principle (Martin 1994). For each time interval during application of lateral load on spudcan, the corresponding loads, displacements and normalized variables can be calculated. Owing to the fact that bending moment dominates the response under combined loads, as can be seen from the following analysis, the curve in normalized moment versus spudcan rotation will be taken as the governing curve
to extract yield points. Eleven tests are analyzed based on this principle.

One of them (sample 230508-2d) will be adopted as an example to describe the progress of analysis. The normalized data $M/RV_{L0}\cdot vs.R.\theta$ is plotted in Fig. 5-9. This relationship can be fitted with 5th order polynomial function following the pattern,

$$y = a_1x^5 + a_2x^4 + a_3x^3 + a_4x^2 + a_5x + a_6$$ (5.4)

where $y$ denotes $M_i/RV_{L0}$ and $x$ denotes the multiplification of $R$ and $\theta_i$. $M_i$ is initialized bending moment, $R$ is the radius of spudcan, $V_{L0}$ is ultimate bearing capacity of spudcan at certain depth, $\theta_i$ is initialized rotation angle of spudcan. (due to the limitation of the data logger, some of the variables can not be set to zero at the start of the experiments, so the initial value have to be subtracted from revelant collected data to obtain the true response. This is called “initialization”)

The curvature of a curve can be defined as:

$$k = \frac{|y'|}{(1 + y^2)^{3/2}}$$ (5.5)

Differentiating equation (5.4) and substituting into equation (5.5), one will get

$$k = \frac{|20a_1x^3 + 12a_2x^2 + 6a_3x + 2a_4|}{\left(1 + \left(5a_1x^4 + 4a_2x^3 + 3a_3x^2 + 2a_4x + a_5\right)^2\right)^{3/2}}$$ (5.6)

The plot $k$ versus $R\cdot\theta_i$ is shown in Fig. 5-10 where the maximum curvature value is found to be -0.062. Then, relevant data, such as $V/V_{L0}$, $M/RV_{L0}$, $H/V_{L0}$ will be found in processed data for this case. Following this principle, the rest of the tests are analyzed and the polynomial fitted curve, curvature plot, and normalized bending moment versus horizontal force plot are displayed in Fig. 5-12 to Fig.
5-28. The yielding points obtained from these tests are listed in Table. 5-2.

### 5.4 Yield surface and yield points

The yield surface described by equation (2.66) together with parameters recommended by Martin (1994) are plotted to investigate the yield points obtained using the above-mentioned methods. As the yield function is normalized by the ultimate bearing capacity which governs the size of the yield surface, the contour determined by this yield function would be the same.

Conveniently the yield surface is divided into 20 segments along $V/V_{L0}$ axis where each segment corresponds to a unique $V/V_{L0}$ value. For every given segment, 24 sections in polar coordinates are divided in $M/2RV_{L0}$ versus $H/V_{L0}$ plane, and would evenly generate 24 points of the yield surface in every deviator plane. This process is programmed using visual basic language and the results corresponding to relevant $V_{L0}$ value can be obtained.

The yield surface and yield points are plotted on the same planes as shown in Fig. 5-29 to Fig. 5-32. The points shown in these figures are corresponding to the test numbers in the legends where $t0$ means at time zero. The exact values of these points could be found in Table. 5-2, referring to corresponding test numbers. Only one point numbered 270408-1.5d lies farther outside the yield surface. Others can almost meet the yield surface predicted by the yield function, even though some of them show a small deviation.
5.5 Verification conclusions

In this section, the similarity of Cam Clay model and strain-hardening force-resultant model is briefly explained. It can be seen that the flexibility ratio of penetration and unloading for the spudcan is predictably large enough to meet the requirement of simulating yield surface with two segments which represent yield locus starting from lower to higher unloading ratio. Eleven tests with different unloading ratios and penetration depths were conducted. The yield points of these tests are obtained using the greatest curvature principle and put into the specified yield surface. It is found that the yield points fit well with the surface except one strange point lying far from it. These test results verify the accuracy of the yield surface of strain hardening force-resultant model.
Table 5-2: The calculation value of yield function for different tests.

<table>
<thead>
<tr>
<th>Test date</th>
<th>Penetration</th>
<th>V/V o</th>
<th>H/V o</th>
<th>M/RV o</th>
<th>R.θ</th>
<th>a</th>
<th>f</th>
<th>Model parameter for clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>23-May-08</td>
<td>1D</td>
<td>0.8</td>
<td>-0.011</td>
<td>-0.119</td>
<td>-0.06</td>
<td>0.329</td>
<td>0.075ho</td>
<td>0.127</td>
</tr>
<tr>
<td></td>
<td>2D</td>
<td>0.84</td>
<td>-0.003</td>
<td>-0.067</td>
<td>-0.062</td>
<td>0.359</td>
<td>-0.138mo</td>
<td>0.083</td>
</tr>
<tr>
<td>24-May-08</td>
<td>1D</td>
<td>0.54</td>
<td>-0.006</td>
<td>-0.164</td>
<td>-0.13</td>
<td>0.224</td>
<td>-0.005β1</td>
<td>0.764</td>
</tr>
<tr>
<td></td>
<td>1.5D</td>
<td>0.73</td>
<td>-0.05</td>
<td>-0.122</td>
<td>-0.12</td>
<td>0.285</td>
<td>-0.066β2</td>
<td>0.882</td>
</tr>
<tr>
<td>27-April-08</td>
<td>1D</td>
<td>0.47</td>
<td>-0.005</td>
<td>0.158</td>
<td>0.1</td>
<td>0.224</td>
<td>-0.075e1</td>
<td>0.518</td>
</tr>
<tr>
<td></td>
<td>1.5D</td>
<td>0.51</td>
<td>-0.014</td>
<td>0.242</td>
<td>0.12</td>
<td>0.223</td>
<td>1.226e2</td>
<td>1.18</td>
</tr>
<tr>
<td>20-August-08</td>
<td>0.5D</td>
<td>0.486</td>
<td>-0.022</td>
<td>-0.144</td>
<td>-0.034</td>
<td>0.223</td>
<td>-0.281</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1D</td>
<td>0.487</td>
<td>-0.012</td>
<td>-0.142</td>
<td>-0.018</td>
<td>0.223</td>
<td>-0.292</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.5D</td>
<td>0.492</td>
<td>-0.011</td>
<td>-0.144</td>
<td>-0.019</td>
<td>0.223</td>
<td>-0.267</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2D</td>
<td>0.498</td>
<td>-0.012</td>
<td>-0.15</td>
<td>-0.019</td>
<td>0.223</td>
<td>-0.204</td>
<td></td>
</tr>
</tbody>
</table>

a is defined in equation (2.67), f is the value of yield function equation (2.66)
Fig. 5-1: Undrained triaxial tests on a Modified Cam-Clay with different $\lambda / \kappa$ (Martin 1994)
Fig. 5-2: Sideswipe test results for a flat circular footing on sand (Tan 1990)

[a] Initial penetration of footing, followed by sideswipe test from high $V/V'_o$

[b] Sideswipe test from low $V/V'_o$

Fig. 5-2: Sideswipe test results for a flat circular footing on sand (Tan 1990)
Fig. 5-3: Unloading-reloading response of test xj0201-1 at (a) 5.3m and (b) 9.8m penetration depth

Fig. 5-4: Curve fitting of unloading-reloading behavior for test xj0201-1 at 5.3m penetration
Fig. 5-5: Curve-fitted stability No. in accordance with Meyerhof’s data (Meyerhof 1972)

Fig. 5-6: Spudcan penetration and unloading behavior for test xj0201-1 at 5.3m penetration
Fig. 5-7: Curve-fitting of unloading-reloading behavior for test xj0201-1 at 9.8m penetration

Fig. 5-8: Unloading-reloading response of test xj0201-1 at 9.8m penetration
Fig. 5-9: Fitted $\frac{M_i}{RV_{Lo}}$ vs. $R \cdot \theta_i$ of test 230508-1d

Fig. 5-10: Curvature vs. $R \cdot \theta_i$ for test 230508-1d
Fig. 5-11: $H/V_{L0}$-$M/RV_{L0}$ for the case of $V/V_{L0}=0.8$ at penetration around 1D of test xj0201-01

Fig. 5-12: Fitted $M/RV_{L0}$ vs. $R \cdot \theta_i$ of test 230508-2d
Fig. 5-13: Curvature vs. $R \cdot \theta_i$ of test 230508-2d

Fig. 5-14: $H_i/V_{oi}$ vs. $M/RV_{L0}$ for the case of $V/V_{L0} = 0.85$ at penetration around 2d of test xj0201-01-230508
Fig. 5-15: Fitted $M_i/RV_{Lo}$ vs. $R \cdot \theta_i$ of test 240508-1d

Fig. 5-16: Curvature vs. $R \cdot \theta_i$ of test 240508-1d
Fig. 5-17: Fitted $M_i/RV_{Lo}$ vs. $R \cdot \theta_i$ of test 240508-1.5d

Fig. 5-18: Curvature vs. $R \cdot \theta_i$ of test 240508-1.5d
Fig. 5-19: $M/RV_{Lo}$ vs. $H/V_{Lo}$ of test 240508-1.5d

Fig. 5-20: $M/RV_{Lo}$ vs. $R \cdot \theta_{i}$ of test 270408-1d
Fig. 5-21: Curvature vs. $R \theta_i$ of test 270408-1d

Fig. 5-22: $M/RV_{i0}$ vs. $H/V_{i0}$ of test 270408-1d
Fig. 5-23: Fitted $\frac{M_i}{RV_{L_o}}$ vs. $R \cdot \theta_i$ for test 270408-1.5d

Fig. 5-24: Curvature vs. $R \cdot \theta_i$ of test 270408-1.5d
Fig. 5-25: M/RV\textsubscript{Lo} vs. H/V\textsubscript{Lo} plot of test 270408-1.5d

Fig. 5-26: Fitted M/RV\textsubscript{Lo} vs. R·θ\textsubscript{i} plot of test 270408-2d
Fig. 5-27: Curvature vs. $R \cdot \theta_i$ plot of test 270408-2d

Fig. 5-28: $M/RV_{L0}$ vs. $H/V_{L0}$ plot of test 270408-2d
Fig. 5-29: 3D view of yield surface and yield points

Fig. 5-30: M/2RV_{L0} vs. H/V_{L0} plot of yield surface and yield points from tests
Fig. 5-31: $M/2R_{VL_0}$ vs. $V/V_{L_0}$ for yield points from tests

Fig. 5-32: $V/V_{L_0}$ vs. $H/V_{L_0}$ plot of yield surface from Martin and yield points from tests
6 Bearing capacity variation of spudcan due to consolidation

6.1 Introduction

In this chapter, the calculated results using several bearing capacity theories, namely, API’s (2002) method, SNAME (2002) method, and Houlsby & Martin’s (2003) method, will be compared with the centrifuge test results. The more accurate theory will be adopted to obtain the ultimate bearing capacity of spudcan before soil consolidation. The variation of bearing capacity of the spudcan during soil consolidation will be correlated with relevant parameters, and a relationship of bearing capacity with time will be generalized. To verify the effectiveness of this generalization, the yield points normalized by the initial and time-dependent bearing capacities will be plotted on the Oxford yield surface (Martin, 1994) so that the applicability of the strain-hardening model under partially drained condition can be examined in this study.

6.2 Determination of bearing capacity using existing theories

In existing force-resultant models, whether they obey associated flow-rule or non-associated flow-rule, the bearing capacity is the primary parameter used to determine the yielding surface and the hardening rule. This implies that bearing
capacity would be an inseparable part of fixity study.

Another reason to do this study is to validate the degree to which the bearing capacity from direct tests and the one calculated with existing bearing capacity theories using T-bar data matches with each other. Three existing classical theories elaborated in Section 2.5, namely, SNAME (2002), API (2002), Houlsby & Martin (2003) have been utilized in this analysis. Note that API's and SNAME (2002) recommended methods are applicable to uniform soil, the average undrained shear strength around the maximum area of the spudcan is approximately taken as the input $C_u$ at designated depth. This is also consistent with the recommendation of SNAME (2002).

These methods are programmed in Fortran language. The flowchart is shown in Fig. 6-1. The backflow effect and spudcan shape are taken into account. The profile of linear shear strength soil and in-situ soil shear strength file can be input into the program. The outcomes will be used to compare with the measured spudcan penetration curves. The equation of linear increase in shear strength of the soil can be expressed as, $C_u = a + bz$

where $C_u$ is undrained shear strength, $a$ is a constant, $b$ is the gradient of the linearized soil line, and $z$ is the depth.

Several test results including tests xj0201, xj0301 and xj0302 are examined. The shear strength profile of clay measured by T-bar at 100g is plotted and fitted with linear line along the penetration depth for every test (see Fig. 6-2, Fig. 6-5, Fig. 6-8). Then, the bearing capacity calculated using three existing methods is
combined into the plot of measured bearing capacity versus depth under linear or in-situ soil shear strength. Let us take test xj0201 as an example. The $C_u$ versus depth profile is plotted in Fig. 6-2 with fitted gradient 3.149 which indicates that the soil is over consolidated. The preloading during consolidation can be clearly seen at the initial stage of penetration curve whose slope is apparently flatter than the soil underneath. The results from linear shear strength soil profile and in-situ soil profile are shown in Fig. 6-3 and Fig. 6-4. The two comparison plots illustrate the same trend; that is, API method will predict larger bearing capacity, while SNAME(2002) will produce the least value. But the later plotted data for normally consolidated clay in Fig. 6-5 to Fig. 6-10 show a different behavior, the API’s prediction is apparently larger than measured data. This indicates that the OCR may have an important influence on the soil bearing capacity. To get consistent results from all tests, it is decided to avoid the overconsolidation effects for this complex study and focus on normally consolidated clay.

The results from Houlsby & Martin’s method is better fitted with the measured data, while the SNAME’s method conservatively predicts 60% of real value, API’s method is 10~20% larger than the measurement for the normally consolidated clay case.

Thus, it is concluded that Houlsby & Martin’s method would be more appropriate than the other approaches when initial bearing capacity of normally consolidated clay with linear strength profile is analyzed.
6.3 Yield points normalized by initial undrained bearing capacity

In section 5, tests were done to verify the Oxford yield surface, but these tests were all conducted under undrained condition (apply lateral load immediately after penetration). For jack-up spudcan site assessment, the undrained shear strength provided by soil investigation companies can only reflect the soil property at the moment the spudcans are installed. Thus, the bearing capacity estimated with these undrained shear strength will be the undrained bearing capacity. How the initial ultimate bearing capacity will affect the soil yield after consolidation remains a question.

Following the principle stated in Section 6.2, the ultimate bearing capacity of the clay under undrained condition in clay at a particular depth, $V_{Lo}$, will be determined. Similar to Section 5, the yield points normalized by $V_{Lo}$ can be obtained. These data are summarized in Table. 6-1.

To consider the time effects, these data are classified and plotted in Fig. 6-15 to Fig. 6-20 according to the centrifuge consolidation time, namely, $t=0$, 0.5 hour, 1 hour, 1.5 hours, 2 hours and 3 hours corresponding to prototype time of 0, 0.57 year, 1.14 year, 1.71 year, 2.28 year, 3.42 year.

In this section, all the yield points are the values normalized by $V_{Lo}$, namely, $V/V_{Lo}$, $H/V_{Lo}$, and $M/2RV_{Lo}$ and the yield points are given in Table. 6-1. As shown in Fig. 6-11~Fig. 6-14, the yield points of consolidating clay are all scattered outside the yield surface. This means strength gain after consolidation is not
captured by the model using undrained parameters. The yield points after different soil consolidation times will be displayed independently in order to make the time effects apparent. These points shown in Fig. 6-15~Fig. 6-20 correspond to centrifuge consolidation time $t=0$, 0.5hour, 1hour, 1.5hour, 2hours and 3hours respectively. These figures indicate a uniform tendency, that is, the more time the spudcan is operating in one place, the farther the distance between the actual yield points and Oxford yield surface will be.

This section proves that the strength gain phenomenon due to consolidation of the clay around spudcan exists and it is no longer correct to use the initial undrained bearing capacity in the strain-hardening force resultant model directly since it does not reflect the soil behavior after some consolidation has occurred. The true ultimate bearing capacity should be determined so that the yield points normalized by this true capacity after consolidation are still lying on the yield surface. This conclusion leads us to find an effective way to incorporate the time effects into this force-resultant model so that it is still applicable after consolidation.

### 6.4 Bearing capacity variation after some consolidation

When the spudcan penetrates to a certain depth, the bearing capacity can be determined by the above-mentioned methods. Pore pressure dissipation due to soil consolidation will cause the soil around spudcan to regain strength.

For all the tests from different batches, the value of yield function is zero for
every yield point. This means the yield points should always lie on the yield surface, even though experimentally some of yield points show some deviations. Based on this principle and coefficients provided by Martin (1994), a visual basic code is compiled to find the correct bearing capacity value, $V_{tv}$, at designated testing time. The results produced by this method are given in Table. 6-1. These points normalized by correct ultimate bearing capacity calculated by above-mentioned method are plotted in Fig. 6-21~Fig. 6-24. It can be seen that the yield points conducted on different batches at different times are all lying on the yield surface which is obtained by normalizing the force-resultant model with $V_{tv}$.

### 6.5 Bearing capacity variation with time

Since the correct bearing capacity of the spudcan after some consolidation has taken place cannot be readily measured or determined theoretically, it is proposed that this is estimated empirically as follows.

A new parameter $\xi$ is introduced,

$$\xi = \frac{V_t}{V_{t0}} \tag{6.1}$$

where $V_t$ is the time-dependent bearing capacity of spudcan.

Thus, $\xi$ always starts from 1. The processed data related to $\xi$ are displayed in Table. 6-2.

Hyperbolic fitting is applied to describe the relationship of bearing capacity multiplier $\xi$ and time. It can be presented as follows,
\[ \xi = \frac{t}{c + dt} + 1 \]  \hspace{1cm} (6.2)

Firstly, the above equation is re-arranged as

\[ \frac{t}{\xi - 1} = c + dt \]  \hspace{1cm} (6.3)

The linear fitted line for \( \frac{t}{\xi - 1} \) and \( t \) is shown in Fig. 6-25. The coefficients \( c, d \) are fitted as 0.451, 1.667, respectively. The formula (6.3) can be shown as

\[ \xi = \frac{t}{0.451 + 1.667t} + 1 \]  \hspace{1cm} (6.4)

The points obtained from tests and fitted hyperbolic line are shown in Fig. 6-26. It can be seen that the fitted line matches reasonably well with the average value of data, even though the data are scattered to some extent.

6.6 Yield points normalized by time-dependent bearing capacity

In Section 6.3, the yield points normalized by initial bearing capacity, \( V_{lo} \), no longer fit well on the yield surface when the soil around the spudcan experiences a period of consolidation. But when the correct bearing capacity \( V_{tv} \) is used, the yield points fitted well on the yield surface. The time-dependent bearing capacity may be estimated with the generalized equation in Section 6.5. How the yield points vary when normalized with time-dependent bearing capacity, \( V_t \), will be studied here.

As mentioned in previous sections, the yield points and initial bearing capacities have been determined. With the correlated equation (6.4) in last section, the
time-dependent bearing capacity, $V_t$, can be calculated with respect to the respective initial bearing capacity. Then, the yield points will be normalized by $V_i$, and plotted in normalized coordinates. The processed data are tabulated in Table 6-3 and plotted in Fig. 6-27–Fig. 6-30. The yield points normalized by time-dependent bearing capacity, $V_t$, are much closer to the Oxford yield surface, comparing with these normalized by $V_{Lo}$ in Fig. 6-11–Fig. 6-14, even though a small number of points still lie outside the yield surface due to the scattering of the test data.

### 6.7 Summary

In this chapter, several bearing capacity theories were analyzed and compared with experimental data. It is found that the method proposed by Houlsby & Martin (2003) is more appropriate to determine the bearing capacity of the spudcan in linearly increasing shear strength profile.

Since the correct bearing capacity variation with time cannot be readily measured or calculated theoretically, the bearing capacity variation with time was empirically determined to find the time-dependent bearing capacity of clay after a period of consolidation.

If the yield points are normalized by the time-dependent bearing capacity, $V_t$, they will tend to be very close to the yield surface. The yield surface of the strain-hardening model still can be applicable if the generalized equation (6.4) is used to estimate $V_t$. 
Table 6-1: Processed test data to obtain bearing capacity variation.

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<th>Test date</th>
<th>penetration-waiting time</th>
<th>V/V&lt;sub&gt;Lo&lt;/sub&gt;</th>
<th>H/V&lt;sub&gt;Lo&lt;/sub&gt;</th>
<th>M/RV&lt;sub&gt;Lo&lt;/sub&gt;</th>
<th>R.θ</th>
<th>f yield function</th>
<th>V&lt;sub&gt;Lo&lt;/sub&gt; (kN)</th>
<th>M/RV&lt;sub&gt;Lo&lt;/sub&gt;/R.θ</th>
<th>stiffness increment</th>
<th>M/R/R.θ</th>
<th>V&lt;sub&gt;r&lt;/sub&gt;</th>
<th>kN</th>
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Table. 6-2: Data processing for fitting of bearing capacity with time

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<th>$t(\xi$ -1)</th>
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<th>dep constime</th>
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Table 6-3: Summary table of yield points normalized by initial bearing capacity, $V_{Lo}$, and time-dependent bearing capacity, $V_t$.

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<th>Equi time (year)</th>
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<th>$H/V_{Lo}$</th>
<th>$M/2RV_{Lo}$</th>
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<td>-0.045</td>
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<td>-0.109</td>
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<td>H/V_{L0}</td>
<td>M/2R V_{L0}</td>
<td>Centrifuge time</td>
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Notes:
1. The first part of sample No. is testing date, mid-part is penetration depth, last part is spudcan standing time.
2. Centrifuge is in 100g.
Fig. 6-1: Flowchart of bearing capacity programming

1. Vesic method
   - Input data
   - Calculation
   - Output file
   - Analysis with cavity
     - No
     - Yes: Analysis with backflow
       - No
       - Yes: The end?
         - No
         - Yes: Terminate

2. Skempton's method
   - Soil type
     - clay
     - sand
     - Input data
     - Soil strength profile
     - Every segment of depth
     - Spudcan dimension
     - Analysis with cavity
       - No
       - Yes: Analysis with backflow
         - No
         - Yes: The end?

3. Houlby&Martin's method
   - Input data
     - linear soil
     - Soil strength profile
     - Every segment of depth
     - Backflow
       - No
       - Yes: Analysis with backflow
         - No
         - Yes: The end?

The end? Yes: Terminate
Fig. 6-2: Cu vs. penetration measured with Tbar from test xj0201-230508

\[ Cu = 0.776 + 3.149d \]

Fig. 6-3: Vertical bearing capacity comparison between measurement and three theoretical results for test xj0201-230508
Fig. 6-4: Bearing capacity comparison between measured data and three theories from real depth vs. Cu data for test xj0201-230508

Fig. 6-5: Cu vs. penetration measured with Tbar from test xj0301-180708
Fig. 6-6: Bearing capacity comparison based on different theory under linear soil assumption for test xj0301-180708

Fig. 6-7: Bearing capacity comparison based on different theory using measured Cu value from test xj0302-180708
Fig. 6-8: Cu vs. penetration measured with Tbar from test xj0302-190708 on clay.

Figure shows a graph with Cu (kPa) on the y-axis and penetration (m) on the x-axis. The equation $Cu = 0.91 + 1.467d$ is shown.

Fig. 6-9: Bearing capacity comparison based on different theory under linear soil assumption from test xj0302-190708.

Figure shows a graph with VLo (kN) on the y-axis and penetration (m) on the x-axis, comparing test data with different theoretical models (API, SNAME(2002), Houlsby&Martin).
Fig. 6-10: Bearing capacity comparison for different theories using measured Cu value from test xj0302-190708

Fig. 6-11: 3D view of Oxford surface and overall yield points normalized by $V_{Lo}$ from centrifuge tests
Fig. 6-12: $M/2RV_{L_0}$ vs. $H/V_{L_0}$ plot of Oxford yield surface and yield points from all the tests ever done.

Fig. 6-13: $M/2RV_{L_0}$ vs. $V/V_{L_0}$ of Oxford yield surface and yield points from all the tests ever done.
Fig. 6-14: $H/V_{Lo}$ vs. $V/V_{Lo}$ plot of Oxford yield surface and yield points from all the tests ever done.

Fig. 6-15: 3D plot of yield points normalized by corresponding bearing capacity, $V_{Lo}$, at $t=0$ hour.
Fig. 6-16: 3D plot of yield points normalized by corresponding bearing capacity, $V_{Lo}$, after 0.5 hour consolidation in centrifuge

Fig. 6-17: 3D plot of yield points normalized by corresponding bearing capacity, $V_{Lo}$, after 1 hour consolidation in centrifuge
Fig. 6-18: 3D plot of yield points normalized by corresponding bearing capacity, $V_{Lo}$, after 1.5 hours consolidation in centrifuge.

Fig. 6-19: 3D plot of yield points normalized by corresponding bearing capacity, $V_{Lo}$, after 2 hours consolidation in centrifuge.
Fig. 6-20: 3D plot of yield points normalized by corresponding bearing capacity, $V_{Lo}$, after 3 hours consolidation in centrifuge.

Fig. 6-21: 3D plot of yield points normalized by corresponding true bearing capacity, $V_{tv}$.
Fig. 6-22: $M/2Rv_v$ vs. $V/V_{tv}$ of Oxford yield surface and yield points normalized by true ultimate bearing capacity, $V_{tv}$

Fig. 6-23: $M/2Rv_v$ vs. $H/V_{tv}$ of Oxford yield surface and yield points normalized by true ultimate bearing capacity, $V_{tv}$
Fig. 6-24: $H/V_{tv}$ vs. $V/V_{tv}$ of Oxford yield surface and yield points normalized by true ultimate bearing capacity, $V_{tv}$.

Fig. 6-25: Linear fitting of $t/(\xi - 1)$ with $t$ for the study of bearing capacity variation.
Fig. 6-26: Strength multiplier variation under partial drained condition

\[ \xi = 1 + \frac{t}{(0.451 + 1.667t)} \]

Equi time (Year)

Fig. 6-27: 3D plot of yield points normalized by time-dependent bearing capacity, Vt.
Fig. 6-28: M/2RV_t vs. V/V_t plot of Oxford yield surface and yield points normalized by time-dependent bearing capacity, V_t.
Fig. 6-29: $M/2RV_t$ vs. $H/V_t$ plot of Oxford yield surface and yield points normalized by time-dependent bearing capacity, $V_t$.

Fig. 6-30: $H/V_t$ vs. $V/V_t$ plot of Oxford yield surface and yield points normalized by time-dependent bearing capacity, $V_t$. 
7 Conclusions

As has been described at the start of this thesis, this study is mainly to verify the existing rotational stiffness theories, the bearing capacity theories, the yield surface of the strain-hardening model under undrained condition, and to determine the partially drained effects on rotational stiffness, bearing capacity of the spudcan and its effect on the yield surface of the force-resultant model using centrifuge tests. Most of the attention was put on the rotational stiffness and ultimate bearing capacity variation with time within the framework of the strain-hardening force-resultant model. The conclusions are as follows.

1) The rotational stiffness of six cases tested in the centrifuge was presented. The classical elastic theories, SNAME (2002) and Bell’s FEM, were applied to assess corresponding rotational responses. Bell’s FEM result was determined as the basis of initial rotational stiffness, \( k_{ro} \).

2) A hyperbolic relationship of spudcan rotational stiffness variation with consolidation time is suggested as follows:

\[
\beta = \frac{t}{a + bt} + 1
\]

where \( \beta = \frac{k_r}{k_{ro}} \) is the normalized rotational stiffness.

\( k_r \) is the rotational stiffness of the spudcan at time \( t \),

\( k_{ro} \) is the rotational stiffness of the spudcan immediately after penetration.

\[ a = 0.31 - 2.314n + 6.018n^2 \]
\[ b = 0.597 - 0.095n \]

n is unloading ratio of the spudcan, \( n = \frac{V}{V_{Lo}} \).

This relationship can be incorporated into strain-hardening model when the load path is needed under partially drained condition.

3) Results of eleven centrifuge tests under undrained condition were presented and their yield points were assessed with the yield surface derived by Martin based on small scale models under 1g condition. It is shown that these points lie on the yield surface and fit well with the surface except for one point which deviated from it for some unexpected reasons. This validated the feasibility of yield surface of the force-resultant model for prototype spudcans partly.

4) Bearing capacity of the spudcan was assessed with classical theories. Although the theoretical results do not match exactly with the measurement, there is some measure of agreement. It is shown that the outcome from Houlsby & Martin (2003)’s method is closer to the measured data. The method adopted by API, is less conservative, while the SNAME (2002)’s recommendation is more conservative.

5) The ultimate bearing capacity of spudcan in clay under partial drained condition is investigated. An empirical expression reflecting the bearing capacity variation with time is fitted for the linear increasing shear strength profile kaolin clay as follows:

\[ \xi = \frac{t}{c + dt} + 1 \]  \hspace{1cm} (7.2)
where $\xi$ is bearing capacity multiplier, defined as $\xi = \frac{V_t}{V_{Lo}}$

$V_t$ is the ultimate bearing capacity of the spudcan at time $t$,

$V_{Lo}$ is ultimate bearing capacity of the spudcan immediately after penetration.

c, d are fitted as 0.451, 1.667, respectively for kaolin clay.

6) The yield points normalized by the initial bearing capacity $V_{Lo}$ and time dependent bearing capacity $V_t$ are plotted on Oxford yield surface by Houlsby and Martin (1994). It is found the points normalized by time-dependent bearing capacity can better match with the yield surface much better than the ones normalized by $V_{Lo}$. This is because the clay gains strength after consolidation and this effect should be reflected on yield surface of the force-resultant model. However, the bearing capacity of the spudcan after consolidation cannot be readily measured or calculated with existing theories. An empirical method was proposed to determine the bearing capacity of the spudcan which varies with consolidation time to consider the time effects in force-resultant model without loss of its original definition.

7.1 **Recommendations for future work**

1) This research is done in kaolin clay using centrifuge tests. More tests on other types of soils may be done to verify the findings in this context. And, how the drainage path affect the spudcan response need to be further
investigated.

2) Only rotational stiffness under partial drained condition is investigated in this study. It may provide a more comprehensive view if vertical and horizontal stiffness are also studied in future.

3) In this study, only the yield surface of force-resultant model is investigated. The hardening rule after consolidation may be a good topic for further study.

4) All the tests whether they are drained or undrained are tested under static load condition. The inertia force and damping effects are not considered and this may not reflect the real behavior of jackup-soil interaction. Force-displacement, pore pressure dissipation with consideration of dynamic effects can be a good topic to study in future.
References

layer," Ph.D, National University of Singapore, Singapore.


