Jack-Up Reinstallation near Existing Footprints

By

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BEng. MEng

This thesis is presented for
the degree of Doctor of Philosophy at

THE UNIVERSITY OF WESTERN AUSTRALIA

Achieving International Excellence

School of Civil and Resource Engineering
Centre for Offshore Foundation Systems

Dec 2011
DECLARATION FORM

This thesis contains published work and/or work prepared for publication, which has been co-authored. This bibliographical details of the work and where it appears in the thesis are outlined below


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I hereby declare that, except where specific reference is made to the work of others, the contents of this dissertation are original and have not been submitted in whole or in part for consideration for any other degree of qualification at this, or any other, university.

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Dec 2011
ABSTRACT

Jack-ups are self-elevating mobile units operating in offshore oil and gas fields situated in water depths up to 120m or so. A typical jack-up unit consists of a floatable hull platform and three independent retractable truss-work legs each resting on a spudcan footing. The spudcans are typically between 10 to more than 20m in diameter. During the removal of a jack-up unit from a site, the extraction of spudcans leave significant seabed depressions. In the offshore industry, these are referred to as “footprints”. Jack-up units often have to return to the same site and be installed near or into existing footprints. This is a problematic operation because the spudcan located near the footprints is subjected to eccentric and/or inclined loading conditions. This can lead to structural failures within the jack-up legs. Furthermore, movements can occur in the spudcan and jack-up legs, with potential risks that the jack-up hull may hit the fixed platform. The jack-up reinstallation problem is perceived by the industry as a significant problem with impact on time, costs and safety of structures and personnel.

Jack-up reinstallation response is relatively complex and in this study three controlling parameters were identified: the footprint’s geometry, the footprint’s soil properties and the structural properties of the jack-up unit. This study reports a comprehensive series of experimental investigation on each of these three parameters, first in isolation and finally in combination. The motivation is that with an improved knowledge of the role of these parameters, prediction method can be developed and the effectiveness of mitigation measures can be assessed, ultimately reducing the failure rate of jack-up reinstallation.

The first part of the study was conducted in the drum centrifuge and idealised footprint cavities were cut within a clay sample. This eliminated the effect of variation of undrained shear strength in a real footprint and the isolated effect of footprint geometry was investigated. The footprint geometry was clearly identified as the controlling parameter on the horizontal force and moment observed during reinstallation between the level of the touchdown and the footprint toe. The peak reinstallation response was particularly sensitive to the increase in footprint size (deeper footprint with steeper angle) and the ratio between the spudcan and footprint diameter. This soil failure mechanisms associated with the peak reinstallation responses were also revealed using PIV techniques.

With consistent test data and an improved understanding of the soil failure mechanism, a simple predictive method was developed to allow industry a “first-pass” estimate of the loads on a spudcan during a reinstallation. Only two inputs parameters are required: the undrained shear strength of undisturbed soil and the footprint geometry. Comparison of the prediction with experimental data confirmed that the method performed satisfactorily, with a slight over-estimation. The method is only applicable for the prediction of reinstallation response shallower than the depth of two times the footprint toe level.
In the second part of this study, a real-time hybrid testing method was developed to model the reinstallation of a full jack-up unit. This allowed the effect of structural properties to be evaluated for the first time. The critical element of the reinstalling jack-up leg was modelled physically and it was connected to a new actuator specifically designed for the vertical, horizontal and angular movements. The remainder of the jack-up unit was modelled numerically. The numerical model interacted with the physical model through a real-time control algorithm to model the response of the full jack-up unit.

When a full jack-up model was reinstalled near a real footprint, the peak reinstallation loads reduced significantly compared to the case of single jack-up leg model. During the reinstallation, the footing initially slid and rotated towards the footprint centre, before returning to its original position with further penetration. The reinstallation response was highly influenced by the strength variation near the maximum depth of the first jack-up installation. The strong soil layer in the real footprint pushed the footing away from the footprint centre and induced a second peak horizontal force and moment during relatively deep reinstallation. Both the peak horizontal force and the moment reduced non-linearly with increasing jack-up flexibility. A simplified method was developed to account for the reduction in peak loads with jack-up flexibility.
ACKNOWLEDGEMENTS

First of all, I would like to express my gratitude and appreciation to my supervisors for their guidance. Thanks Professor Mark Cassidy who guided me with great patience and enthusiasm and given me invaluable advice throughout the course of this study. Thanks Associate Professor Christophe Gaudin for the expert advice on the trickier aspects of centrifuge testing. In addition, I would like to thank to Professor Mark Randolph who provided me with the opportunity to undertake the study at COFS. Thanks for making my last 4 years of study at the University of Western Australia such a wonderful and memorable journey.

I would like to acknowledge the assistance and inspiring discussion from Professor Dave White on PIV analyses. The advice from Dr. Okky Purwana on the industrial practice is also greatly appreciated.

Financial support provided by Australian Post-graduate Scholarship, William Lamden Owen Scholarship and Ad-Hoc Top-Up Scholarship are gratefully acknowledged.

I am particularly grateful to the great technicians in the centrifuge team. Thanks to Bart Thompson and Don Herley for not only their assistance, but also their cheer and encouragement throughout the centrifuge testing programme. I would also like to express my appreciation to the “VHM actuator team”: Phil Hortin and Shane De Catania for turning a loose idea into something solid. Thanks also for willing to have the discussion in the café (I will miss the great coffee break!). I wish to thank all the other brilliant technicians including Turan, Dave, John, Khin, Frank, Neil and Alby. Sorry for all those “incidents” and thanks for the rescue works. Thanks also for helping me to pick up the “true Aussie accent”.

My sincere thanks go to all the fellow colleagues in COFS who have been making COFS an excellent place for research. I wish to thank members of the jack-up team: Britta, Youhu and Shazzad for the fruitful discussion. I would also like to express my appreciation to Edmond, Anjui, Kok Kuen and Han Eng for their help since my first arrival in Perth. Thanks also to the colleagues and visitors: Yue, Yifei, Zack, Xu, Indranil, Stefanus, Fauzan, Youssef, Kevin, Long, Kar Lu and Cheng Ti. Not forget to thank members of the COFS/CIVIL swimming team: Don (the coach), Amin, Keith and Yuxia; thanks for giving me a convenient excuse to be out of the laboratory or away from the computer for 1 hour everyday.

Thanks also to the administration staff, Monica for her support on the administrative work, also Lisa and Ivan for handling all the purchasing orders.

I would also like to extend my appreciation to Dr. Jack Pappin, Mr. Chu Wing Law, Ms. Ada Chan and my former colleagues in Arup and CLP. Thanks for giving me the opportunities to work on various challenging projects that inspired me to pursue this study.
Finally, I would like to express my sincerest appreciation to my parents who are my powerful role models; and to my sisters Emily, Maggie and Bonnie, my brother Victor and my partner Alex for the continuous support and encouragement.

Last but not least, I would like to thank God for the love and guidance;

“The LORD is my shepherd, I shall not be in want” (23rd Psalm)
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NOTATION

**Roman**

\( a_H \) .............. Factor controlling the variation of inclination angle with depth
\( a_M \) .............. Factor controlling the variation of eccentricity with depth
\( B \) ............. Width of strip foundation
\( C \) ............. Flexibility matrix
\( C \) ............. Combined flexibility
\( c_v \) ............. Coefficient of consolidation
\( d_c \) ............. Depth factor for bearing capacity factor \( N_c \)
\( D \) ............. Diameter of footing
\( D_f \) ............. Diameter of spudcan creating the footprint
\( d_{\text{model}} \) ........ Depth in model scale
\( d_{\text{prototype}} \) ........ Depth in prototype scale
\( e \) ............. Load eccentricity
\( E \) ............. Young’s modulus
\( E_A \) ............. Axial rigidity
\( E_I \) ............. Flexural rigidity
\( F \) ............. Factor of safety against slope instability
\( F \) ............. Sample size of load readings
\( F^* \) ............. Modified factor of safety
\( F_b \) ............. Ratio of inclination angle to maximum inclination angle
\( F_{bf} \) ............. Ratio of eccentricity to maximum eccentricity
\( F_{3D} \) ............. Three-dimensional factor
\( g \) ............. Gravitational force
\( g_c \) ............. Slope correction factor for bearing capacity factor \( N_c \)
\( G \) ............. Shear modulus
\( h \) ............. Horizontal displacement of footing
\( h_{\text{total}} \) ........ Total horizontal displacement at load reference point
\( h_0 \) ............. Horizontal displacement at load reference point due to rotation
\( H \) ............. Horizontal force
\( H_e \) ............. Critical penetration depth
\( I \) ............. Area moment of inertia
\( I_r \) ............. Rigidity index
\( k \) ............. Gradient of undrained shear strength profile
\( \kappa \) ............. Dimensionless strength parameter for non-homogeneous cohesive soil
\( K \) ............. Stiffness
\( K_o \) ............. Static earth pressure coefficient
\( l \) ............. Length of slope
\( L \) ............. Length of footing
\( L \) ............. Length of jack-up leg
\( M \) ............. Moment
N................. Stability number of slope
N................. Scale to earth’s gravity
N_c................. Bearing capacity factor due to cohesion
N_ball............. Resistance factor for ball penetrometer
N_t-bar............ Resistance factor for t-bar penetrometer
N_t-deep .......... T-bar penetrometer resistance factor for deep flow-round failure mechanism
N_t-shallow ...... T-bar penetrometer resistance factor for shallow embedment
N_RF.............. Reduced bearing capacity factor for footing near footprint
q.................. Bearing pressure
q_{max} .......... Maximum bearing pressure
q_{t-bar}......... T-bar penetration resistance
q_u............... Ultimate bearing resistance
RF............... Reduction factor for bearing capacity factor of footing near footprint
R_{su}............. Undrained shear strength ratio
s_c ................. Shape factor for bearing capacity factor N_c
s_u............... Undrained shear strength
s_{um}............. Undrained shear strength at mudline
t.................. Time
V.................. Vertical displacement of footing
V................. Normalised penetration velocity
V................. Vertical force
W................ Spacing between jack-up legs
X_F ................. Footprint diameter
x_{neutral} ........ Point of separation in two-way failure
z_{contact} ......... Depth where footing comes into full contact with the founding soil
z............... Penetration depth
z_F ................. Footprint depth
z_{FC}............... Maximum penetration depth during initial jack-up installation/footprint creation
z_{emax} .......... Depth where eccentricity reaches its maximum value
z_{Hmax} .......... Depth where horizontal force reaches its maximum value
z_{Mmax} .......... Depth where moment reaches its maximum value
z_{max} .......... Maximum penetration depth during jack-up reinstallation
z_s ................. Slope height

Greek

\( \alpha \) .............. Load inclination
\( \beta \) ............... Offset distance between the centre of footprint and centre of reinstalling footing
\( \phi' \) ............... Friction angle
\( \gamma' \) ................. Effective unit weight
\( \eta \) ................. Footprint angle or slope angle
\( \theta \) ................... Angular movement of footing
\( \lambda \) ................. Distance between slope crest and edge of footing
\( \tau_1 \) ................. Operation time factor
\( \tau_2 \) ................. Elapsed time factor
\( v \) ................... Penetration velocity
\( \nu \) ................... Poisson’s ratio

**Superscripts**

\( ' \) ................. Effective stress quantity

**Subscripts**

ball ................. Value related to or measured by ball penetrometer
h ................. Horizontal
i .................. Initial value or value at touchdown level
max ................. Maximum value
max_s ............. Maximum value during shallow depth reinstallation
max_d ............. Maximum value during deep reinstallation
model ........... Model scale
prototype .... Prototype scale
rot ................. Rotational
step ................. Step in real-time control algorithm
t-bar ............. Value related to or measured by t-bar
v ................... Vertical
<table>
<thead>
<tr>
<th>ABBREVIATIONS</th>
<th>Definition</th>
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</thead>
<tbody>
<tr>
<td>2D.............</td>
<td>Two-dimensional</td>
</tr>
<tr>
<td>3D.............</td>
<td>Three-dimensional</td>
</tr>
<tr>
<td>CP.............</td>
<td>Common point in the hybrid numerical and physical jack-up model</td>
</tr>
<tr>
<td>ET.............</td>
<td>Elapsed time</td>
</tr>
<tr>
<td>FE.............</td>
<td>Finite element</td>
</tr>
<tr>
<td>H.............</td>
<td>Horizontal actuator</td>
</tr>
<tr>
<td>LDFE........</td>
<td>Large deformation finite element</td>
</tr>
<tr>
<td>LHS ............</td>
<td>Left hand side of the footing (closer to the footprint centre)</td>
</tr>
<tr>
<td>LRP. ..........</td>
<td>Load reference point</td>
</tr>
<tr>
<td>NC.............</td>
<td>Normally-consolidated</td>
</tr>
<tr>
<td>OC .............</td>
<td>Over-consolidated</td>
</tr>
<tr>
<td>OCR ...........</td>
<td>Over-consolidation ratio</td>
</tr>
<tr>
<td>OT .............</td>
<td>Operation time</td>
</tr>
<tr>
<td>PID .............</td>
<td>Proportional-Integral-Derivative</td>
</tr>
<tr>
<td>PIV .............</td>
<td>Particle image velocimetry</td>
</tr>
<tr>
<td>M .............</td>
<td>Angular actuator</td>
</tr>
<tr>
<td>NUS............</td>
<td>National University of Singapore</td>
</tr>
<tr>
<td>RHS .............</td>
<td>Right hand side of the footing (further away from the footprint centre)</td>
</tr>
<tr>
<td>RPD.............</td>
<td>Rack phase difference</td>
</tr>
<tr>
<td>SNAME......</td>
<td>The Society of Naval Architects and Marine Engineers</td>
</tr>
<tr>
<td>UWA ............</td>
<td>The University of Western Australia</td>
</tr>
<tr>
<td>V.............</td>
<td>Vertical actuator</td>
</tr>
<tr>
<td>Vol.............</td>
<td>Volume of footing</td>
</tr>
<tr>
<td>VHM ...........</td>
<td>Vertical-horizontal-rotational actuator</td>
</tr>
</tbody>
</table>
CHAPTER 1. INTRODUCTION

1.1 PREFACE

Offshore oil and gas fields operation in water depths up to 120m or so are performed from self-elevating mobile units, which are generally referred to as jack-up units. Jack-up units have proven flexibility and cost-effectiveness in field development and drilling operation, making them an essential element of the offshore industry.

This study was undertaken to investigate the behaviour of a jack-up when being installed near an existing footprint on the seabed. The problem is complex in that there is significant interactions between the geometry of the footprint, the strength of the soil and the structural configuration of the jack-up itself. This research was conducted with an aim to improve the understanding on the influence of each of these key parameters in isolation and subsequently in combination. In this study, physical modelling was conducted to provide insights into the behaviour of a jack-up during reinstallation, and to subsequently develop a method to predict the loading acting on the leg.

1.2 JACK-UP UNIT

Jack-ups unit are self-elevating mobile units and are used extensively in the offshore oil and gas industry. Figure 1.1 shows a typical jack-up unit consisting of a floatable platform (hull) and three independent retractable truss-work legs each resting on a spudcan footing. The spudcans are of inverted cone shape commonly in polygonal or circular shape in plan. For jack-up built before the 1980s, the diameter of spudcans were within 5 to 10m (Young et al. 1984). The shape and size of spudcans evolved for the operation of jack-up unit in deeper waters and modern spudcans are typically between 10 to 20m in diameter (Menzies and Roper 2008). The bearing pressure on a spudcan is usually around 200-600kPa. Figure 1.2 shows examples of typical spudcans (after Teh 2007).
Figure 1.1: A typical three legged jack-up unit (after Reardon 1986, Purwana et al. 2005)

Figure 1.2: Examples of Spudcan Footing (after Teh 2007)

Figure 1.3 shows the operation sequence of a jack-up unit. After a jack-up unit is towed to site, installation begins by elevating the hull above the water and lowering the legs into the seabed. The spudcan footings are then preloaded by pumping water into the hull to apply a vertical load that is 50% to 100% greater than the operation load (SNAME 2002). This is to ensure the foundations have sufficient reserve capacity in any extreme storm design event (SNAME 2002). Once preloading is completed, the water ballast is removed and the hull is jacked up to its operation height providing a stable working platform. Modern jack-up units can operate in water depths up to 120m or so. Depending on the nature of project, the operation time of a jack-up unit can be from as little as a few days to as long as a few years (Purwana 2007). During the removal of a jack-up unit, the legs are retracted from the seabed and the jack-up unit is towed away from the site. This leaves a significant seabed depression in the site.
This seabed depression is referred to as a “footprint”. They are usually more than 10m wide and 10m deep, and surrounded with highly disturbed soil (Stewart and Finnie 2001, Cassidy et al. 2009, Gan 2009). Figure 1.4 shows a survey of spudcan footprints on the seabed. With the extensive and more frequent use of jack-up units in the offshore environment, the existence of footprints is common. Berg (2004) presented a statistical study on the distribution of footprint registered in the geotechnical and footprint datasets. Within Shell EP Europe, for instance, there are roughly 1200 footprint points and there are approximately 80 new single footprint points being added to the dataset each year.
1.3 Jack-up Reinstallation Near Existing Footprints

Jack-up units often have to be installed near or into existing footprints in order to drill additional wells, enhance the production of existing wells or even provide a workover platform for maintenance of smaller fixed platform. This is a problematic operation for the offshore industry. The spudcan located near the sloping surface of the footprints with varying soil strength is subjected to eccentric and/or inclined loading conditions. As it penetrates further it is subjected to varying soil strengths. Depending on the flexibility of the jack-up unit, movements can occur as the spudcan slides into the footprint. The displacement and the forces can be in exceedance of the structural capacity and cause damage to the jack-up legs. The situation is even more complex if the jack-up is reinstalled nearby an existing structure, as there is risk that the jack-up hull will move towards and possibly hit the fixed platform. A confidential review of jack-up installation records revealed that a number of jack-ups have come dangerously close to impacting adjacent platform (Stewart and Finnie 2001). The scenario is illustrated in Figure 1.5. The jack-up reinstallation problem is significant as it can lead to significant lost time, cost implications and potential injury to personnel (Sumrow 2002).

Figure 1.5: Jack-up Reinstallation Scenario
In the jack-up literature this problem has been referred to as jack-up workover, spudcan-footprint interaction (S-F-I) and jack-up reinstallation (Stewart and Finnie 2001, Osborne et al. 2006, Gaudin et al. 2007, Dean and Serra 2004, Gan 2009, Cassidy et al. 2009). In this study, this problem will be simply referred to as jack-up reinstallation.

### 1.3.1 Significance of the Jack-Up Reinstallation Problem

Jack-up reinstallation is a known hazard to the offshore industry, though it has attracted more attention from the industry itself then from academic research (Sumrow 2002). Dier et al. (2004) rated incidents relating to uneven seabed/scour/footprint as the second highest rate of jack-up incidents at 15% of the total. Example of incidents include installation of jack-ups Friede & Goldman Mod V and KFEL’s modified Mod V in the North Sea in year 2000 (Hunt and Marsh 2004). In these incidents, the jack-up leg installed near the footprint was damaged during reinstallation. There is also an increasing frequency of incidents related to jack-up reinstallation near footprints. The statistical survey presented by Osborne (2005) indicated that the number of recorded jack-up reinstallation incidents increased by more than a factor of four over the period from 1979-1988 to 1996-2005 (Figure 1.6). Osborne et al. (2006) referred to a confidential study conducted by the industry and suggested that the increasing reliance of jack-up rigs for drilling and workover activities for unmanned and subsea facilities and the expanding jack-up operational area due to increased jack-up capacities are some reasons for this increasing frequency of incidents.
1.3.2 Industrial Practice and Current Guidelines

Careful design and monitoring is required for jack-up reinstallation near existing footprints. During the planning stage, a detailed bathymetry survey can be conducted to confirm the footprint geometry. Although accurate records of previous jack-up operations creating the footprints are extremely useful to access the soil condition, it is often very difficult to retrieve such records and predict changes in soil strength due to the previous installation (Foo et al. 2003). During jack-up reinstallation, the current design guideline SNAME (2002) provides general advice of keeping a minimum of one spudcan diameter between the edge of the spudcan and the edge of the footprint, with the warning that a larger offset distance may be required for softer soil. However, in many operations, such as working over an existing platform, this minimum offset distance cannot be achieved due to the geometric constraints. There is a need for more substantial guidance.

In a qualitative study of mitigation methods, SNAME (2002) recommends to carefully position the jack-up with one footing over a footprint and the others in undisturbed soil. Common industrial practice is to orient the jack-up unit so that only one leg is located nearby the footprint. The other legs will firstly be installed to increase the rigidity of the system and help restraining future movement. The leg located nearby the footprint will then be installed with careful monitoring.

Foo et al. (2003) suggested monitoring the rack phase difference to prevent leg bending failure and to monitor sliding of the spudcan. Rack phase difference (RPD) is the difference in elevations between the rack teeth of the chords of any one jack-up leg. After the successful installation of all the legs, the hull will be ballasted to the full preloading level. Additional sliding movement and eccentric load may be induced during the preloading process. Although passive onsite monitoring is useful to identify the problem, it is often very costly and dangerous to rectify the problem on site. A more detailed design guideline is therefore necessary for safe jack-up reinstallation.

1.4 PURPOSE AND ORIENTATION OF THE THESIS

The problem of jack-up reinstallation is relatively complex and it is argued in this thesis that the response is dependent on three parameters: the footprint’s geometry, the footprint soil properties and the structural properties of the jack-up unit (Figure 1.7). Therefore, this study aimed to improve the understanding on the influence of the parameters in isolation and in combination by a comprehensive series of experimental investigation, so that better guideline and more effective mitigation measures could be developed.
In the first part of the study, idealised footprint cavities were formed with minimum disturbance to the soil, so that the role of footprint geometry could be investigated in isolation. Reinstallation tests were performed in the centrifuge to simulate stress conditions equivalent to the offshore environment. Half-footing reinstallation model tests and particle image velocimetry analyses (PIV) were also employed to reveal the soil failure mechanism during reinstallation. By combining the recorded loading from an extensive series of reinstallation tests and the understanding of soil failure mechanisms from PIV tests, a simple method was developed to predict the reinstallation response. The overall motivation is to provide to industry a “first-pass” method to allow a check on the effect of spudcan induced loading on the structural capacity of the overall jack-up. This will help to avoid over-loading in the jack-up unit.

In the second part of the study, a full jack-up unit was considered to investigate the effect of structural properties to the reinstallation responses. The real-time hybrid testing method that was originally developed for dynamic testing of large civil engineering structure, was extended for the modelling of a full jack-up unit in this study. A series of jack-up units with different structural properties were considered to evaluate the effect of structural properties on reinstallation response. The change in maximum loading and movement with jack-up flexibility was quantified. The aim of this work is to allow prediction of reinstallation load for jack-up with different structural properties.

In this study, a real footprint is defined as a footprint created by the installation and extraction of a footing. An idealised footprint cavity is defined as one cut out of the surface by the experimentalist. Figure 1.8 shows the sign convention adopted in this thesis for the footing load and movement. Positive vertical load indicates footing is resisting an upward load from the soil, positive horizontal load indicates the footing is
resisting a horizontal load towards the footprint and a positive moment indicates the footing is resisting a counter-clockwise rotation away from the footprint.

![Diagram](image)

**Figure 1.8: Sign Convention Adopted in This Study**

The outline of the thesis is detailed below:

Chapter 2 is a literatures review, with detailed discussion on the three identified components of the footprint geometry, the footprint soil strength and the structural properties of the jack-up unit. The SNAME guidelines and possible mitigation measures are also reviewed.

Chapter 3 presents the planning, the methodology and the results of footing reinstallation tests conducted in the drum centrifuge at UWA. Idealised footprint cavities with undisturbed soil were considered to identify the geometric effect of a footprint in isolation. A range of idealised footprint cavities with different footprint diameter and footprint depth were considered.

Chapter 4 presents the details of half-footing visualisation tests in the centrifuge. The qualitative examination on the soil failure mechanisms during footing reinstallation near the idealised footprint cavity is presented. PIV analyse results showing precise quantification of soil flow patterns are also compared.

Chapter 5 develops a simple method for the prediction of shallow depth reinstallation response. Critical input parameters were determined by back-analysing the experimental results presented in Chapter 3 and Chapter 4.
Chapter 6 explains the development of a real-time hybrid testing method for the combined physical and numerical modelling that can account for the full jack-up structural properties during a reinstallation test. The history of real-time hybrid testing method was firstly reviewed, followed by a detailed description on the development of a novel vertical-horizontal-rotational actuator and the real-time control algorithm. The calibration and validation test results are also shown.

Chapter 7 presents the details of the real-time hybrid tests conducted on the laboratory floor. Jack-up units with different structural properties were reinstalled near idealised footprints for the investigation of the combined effect of footprint geometry and jack-up structural stiffness on the reinstallation response.

Chapter 8 presents the details of real-time hybrid test in the beam centrifuge at UWA. Jack-up units with different structural properties were reinstalled near real footprints. For the first time these tests are a complete model considering all three governing parameters. The change of load-displacement response with structural properties is presented.

Chapter 9 reports a prediction method to assess the effect of structural properties to jack-up reinstallation response.

Chapter 10 summarises the main findings from this research and provides recommendations for future studies.
CHAPTER 2. LITERATURE REVIEW

2.1 INTRODUCTION

The responses of jack-up reinstallation near footprints are complex and related to a large number of variables. Unfortunately, there is only limited field data available addressing this problem. Previous investigations mainly involved experimental modelling in centrifuges and numerical modelling using the finite element technique. A summary of the key literature is presented in Table 2.1 and Table 2.2. This chapter presents a detailed discussion according to the three critical factors identified for jack-up reinstallation: footprint geometry, footprint soil strength and the structural properties of the jack-up unit. Sections 2.2 to 2.4 cover each issue separately. The current design guidelines of SNAME 2002 and Dier et al. 2004 and possible mitigation measures are also summarised, and discussed in Section 2.5. A more detailed literature review specific to centrifuge modelling techniques, the real-time hybrid testing method, and the development of theoretical solutions are presented in the relevant sections of subsequent chapters.
Table 2.1: Summary of Studies on Jack-Up Reinstallation – Physical Modelling (All Dimensions in Prototype Dimensions)

<table>
<thead>
<tr>
<th>References</th>
<th>Soil Type &amp; Properties</th>
<th>Spudcan</th>
<th>Model Set Up &amp; Variables</th>
<th>Key Findings</th>
</tr>
</thead>
</table>
| Stewart and Finnie (2001)   | OC Clay                | Circular      | 12m                      | 200g  Not Available Fully Fixed                                                                                     | Critical offset distance was at 0.75D  
  Analytical model was presented for predicting horizontal load displacement responses (detailed calculation not available) |
| Teh et al. (2006)           | Sand                   | Circular & Skirted | 125mm                  | 1g  Leg Not Modelled Fully Fixed                                                                                      | There was significant different between spudcan reinstalled near sand slope and footprint in sand |
|                            | OC Clay Overlying Sand | s_c=11.3      |                          |                                                                | Larger horizontal force was generated when installing skirted footing near clay slope |
| Gaudin et al. (2007)        | NC Clay                | Circular (Mod VA) | 15m                      | 250g  3.20MN/m Fully Fixed Free-to-Slide                                                                               | There was significant change to the moment response when free-to-slide connection was adopted |
| Cassidy et al. (2009)       | OC Clay                | Circular (Mod VA) | 18.19m                  | 250g  1.60MN/m  3.20MN/m 12.6MN/m 25.7MN/m Fully Fixed                                                                 | Critical offset distance was between 0.5 to 1.0D |
|                             |                        | Cross (116C)  | 18.3m                    |                                                                | Preload (larger than or equal to 60MN) and leg stiffness did not affect reinstallation responses |
|                             |                        |               |                          |                                                                | Laser profiling of footprint was presented |
| Gan (2009) (tests conducted at NUS) | NC Clay   | Circular  | 6m 8m 10m               | 100g  3.60MN/m 55.1MN/m Fully Fixed                                                                        | Critical offset distance was between 0.5 to 1.0D  
  Parametric study on footprint footing ratio, leg flexibility, preload was presented |
| Gan (2009) (tests conducted at UWA) | NC Clay | Circular (Mod VA) | 14.5m                    | 200g  2.56MN/m Fully Fixed                                                                                           | s_c profiles of footprints in NC and OC material were compared  
  s_c increased with operation time and elapsed time |

# NC Clay refers to normally-consolidated clay and OC clay refers to over-consolidated clay

* Critical offset distance is the offset distance (between the footprint and the footing) where the largest maximum horizontal and moment loads occur
<table>
<thead>
<tr>
<th>References</th>
<th>Soil Type &amp; Properties</th>
<th>Spudcan Dia.</th>
<th>Footprint Dia.</th>
<th>Footprint Depth</th>
<th>Model Set Up &amp; Variables</th>
<th>Leg Bending Stiffness</th>
<th>Key Findings</th>
</tr>
</thead>
</table>
| Jardine et al. (2002)        | Clay                    | s_u =37.5 to 60.0 | 14.3m          | 14.3m          | 5.0m                     | 0.10MN/m             | * Field measurement of a footprint was presented  
* Wish-in-place plane-strain finite element model was adopted to investigate footprint infilled with gravel and sand  
* The horizontal load was lower in footprint infilled with gravel than with sand  
* Neither of the infilling material could reduce the horizontal load to within the structural capacity of jack-up  
* Horizontal force increased with increasing leg bending stiffness |
| Carrington et al. (2003)     | Clay                    | s_u =75.0     | 18.0m          | 24.0m          | 3.9m                     | Leg Bending Stiffness Not Available  
Rotational Stiffness=16000MNm/rad  
Horizontal Stiffness=2MN/m | * Large Strain finite element model was adopted  
* Critical offset distance was at 0.407D  
* Spudcans modified with central tip, skirt and block were considered  
* Skirted spudcan resulted in higher horizontal force in opposite direction |
| Grammatikopoulou et al.      | Clay                    | s_u =31.1 to 44.0 | 18.2m          | 18.2m          | 3.0m                     | Not Available | * Plane-strain finite element model was established for the modelling of infilled footprint gravel  
* Both infilled footprint and in-filled footprint capped with loading platforms were considered  
* Neither of the options could reduce the horizontal load to within the structural capacity of jack-up |
| Foo et al. (2003)            | N/A                     |               |                |                |                          |                      | * RPD monitoring was recommended to avoid damages during jack-up reinstallation |
| Dean and Serra (2004)        | N/A                     |               |                |                |                          |                      | * System response was discussed in detail  
* Conceptual mitigation ideas involving alteration of the structural components of spudcan foundation and/or leg were presented |
2.2 FOOTPRINT GEOMETRICS

2.2.1 Footprint Geometry

There are limited field measurements of footprint geometry, and Jardine et al. (2002) provides the only published field data. Jardine et al. (2002) reported that a footprint of 5m depth was left after the installation and extraction of a Friede & Goldman L780 Mod VI Class jack-up unit in firm to stiff clay. The diameter of the spudcan was 18.18m, but the diameter of the footprint was not presented.

On the other hand, the geometry of footprints has been measured by various researchers using centrifuge modelling. A real footprint is created by the installation and extraction of a single spudcan jack-up leg into clay soil in a centrifuge (Stewart and Finnie 2001, Cassidy et al. 2009, Gan 2009). Under the high gravitational force in the centrifuge, the soil fails and backflows into the cavity, reflecting the field condition. Measurement of its geometry has usually been conducted using laser scanning.

Cassidy et al. (2009) compared the geometry of footprints created by different magnitudes of preload (Figure 2.1). The footprint geometries were measured using laser profiling. The footprints were of a conical shape, with a relatively gentle slope angle. The geometry of footprints created by a maximum installation load of 60MN (maximum bearing pressure \( q_{\text{max}}=231\text{kPa} \)) and 80MN (\( q_{\text{max}}=308\text{kPa} \)) were found to be similar in depth, whereas the 40MN (\( q_{\text{max}}=154\text{kPa} \)) footprint was significantly shallower. A possible explanation for this result is that the maximum penetration depth, \( z_F \), increased to achieve the larger preload in the same soil. The maximum penetration depth (\( z_F \)) was normalised by the footing diameter (\( D \)) as \( z_F/D \). \( z_F/D \) increased from 0.16 to 0.31 and to 0.46 when the maximum installation load increased from 40MN to 60MN and 80MN respectively. During the initial penetration, if the spudcan was buried sufficiently deep in the soil, the final footprint geometry became insensitive to the preload. Figure 2.1 also shows a footprint profile in slightly overconsolidated soil from Gan (2008a). The footprint was created with a maximum installation load of 27MN (\( q_{\text{max}}=104\text{kPa} \)). However the footprint profile was in excellent agreement with the footprints created by Cassidy et al. (2009) with 60MN and 80MN of preload.
Gan (2009) investigated the footprint formed in soils of different undrained shear strengths. Identical preload was applied, and the maximum penetration depth increased with decreasing soil strength. Table 2.3 shows that the footprint diameter increased with reducing soil strength. It could therefore be concluded that the footprint diameter was related to the magnitude of the preload and, hence to the maximum penetration depth during initial penetration. The reported footprint diameter was within the range of 1.0D to 2.1D, and the footprint depth varied between 0.11 and 0.28D.

Table 2.3: Footprints Formed in Soil of Different Strength (after Gan 2009)

<table>
<thead>
<tr>
<th>Soil Strength</th>
<th>$q_{\text{max}}$</th>
<th>$z_{FC}/D$</th>
<th>Footprint Diameter</th>
<th>Footprint Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>$s_u = 50+4.1z$ kPa</td>
<td>460 kPa</td>
<td>0.2D</td>
<td>1.0D</td>
<td>0.21D</td>
</tr>
<tr>
<td>$s_u = 30+3.8z$ kPa</td>
<td></td>
<td>0.5D</td>
<td>1.6D</td>
<td>0.23D</td>
</tr>
<tr>
<td>$s_u = 28+1.65z$ kPa</td>
<td></td>
<td>1.3D</td>
<td>2.0D</td>
<td>0.2D</td>
</tr>
</tbody>
</table>
Cassidy et al. (2009) also reported that the maximum horizontal loads during reinstallation near footprints created with preloading of 60 and 80MN tests were similar, but the maximum horizontal load in the 40MN case was found to be lowest in comparison. However, the dominating factor for the lower peak horizontal load, whether it was an isolated effect or a combined effect of the footprint geometry and soil strength of the footprint, was not identified.

2.2.2 Offset distance

The effect of offset distance between the footprint and reinstallation spudcan has seen considerable experimental investigation. In most, a footprint was first created by penetrating and extracting a model jack-up leg into the soil. The model jack-up leg was then re-penetrated at an offset distance from the footprint centre, simulating the jack-up reinstallation process. The offset distance ($\beta$) is defined as the distance from the centre of reinstalling spudcan to the footprint centre. Shown in Figure 2-2 are typical load-penetration depth responses for all vertical, horizontal and moment loads, as measured in the centrifuge and presented by Cassidy et al. (2009). A reduction in vertical load was clearly shown until reaching $z_{FC}$, where $z_{FC}$ represents the maximum spudcan penetration during footprint creation. The horizontal force and moment at spudcan level increased sharply at the touch-down level. The positive horizontal force pushed the spudcan into the footprint, and the positive moment rotated the spudcan away from the footprint. As penetration continued, both horizontal force and moment kept increasing, but at a slower rate until reaching their peak values at just above the level of $z_{FC}$. The horizontal force and moment then remained almost constant during reinstallation until penetrating near $z_{FC}$, where reduction began.
Figure 2.2: Centrifuge Testing Results of Jack-Up Reinstallation Near a Footprint (after Cassidy et al. 2009)
Figure 2.3 shows the change of normalised horizontal force and moment with offset distance. The critical offset distance is the offset distance where the largest maximum horizontal and moment loads occur. Stewart and Finnie (2001) adopted a single jack-up leg model rigidly connected to the actuator, with the spudcan connected to the other end. The effect of the offset distance was investigated by reinstalling a model jack-up leg at 0 to 2.0D from the centre of the footprint. Based on the maximum horizontal load, it was concluded that the critical offset distance was between 0.5 and 1.0D. Cassidy et al. (2009) adopted a similar testing set-up, but considered a softer clay soil. Offset distances ranging from 0.25 to 1.5D were considered. The peak horizontal force and peak moment were observed at an offset distance of 1.0D. Gan (2009) investigated the jack-up reinstallation response in firm to stiff soil and reported that the peak horizontal force and moment occurred at an offset distance of 0.75D. Gaudin et al. (2007) adopted a free-to-slide jack-up leg actuator connection. Based on the maximum horizontal displacement and moment, the critical offset distance was 1.0D.

Figure 2.3: Critical Offset Distance Between Footprint and Reinstallation (after Stewart and Finnie 2001, Gaudin et al. 2007, Cassidy et al. 2009, Gan 2009)

Carrington et al. (2003) presented the only numerical investigation on the offset distance. Large strain three-dimensional finite element analyses were performed to investigate the jack-up reinstallation response in firm to stiff clay soil. The soil was modelled as elasto-plastic material. A relatively narrow range of offset distances ranging from 0.167 to 0.407D were considered. The horizontal forces generally reduce with increasing offset distance and the critical offset distance was 0.287D. This is lower than the results from the experimental investigations. However, there was only 30% difference in the horizontal force for reinstallation at the three different offset distances, compared to the 300% difference from the experimental investigations (though unfortunately with little explanation as to the reasons why). It is recommended that numerical investigation should consider a wider range of offset distances, particularly extending the range to offset distances larger than 1.0D to confirm the critical offset distance.
The results from the experimental investigations are in good agreement, and the critical offset distance is in the range of 0.5 to 1.0D. However, this finding might only be applicable for the fully-fixed and the free-to-slide jack-up leg actuator connection. It is anticipated that the response may be different if the full jack-up structural stiffness is considered. This requires a more realistic finite stiffness connection to represent the jack-up’s structure and leg-hull connection.

2.2.3 Spudcan Geometry

During jack-up reinstallation, another jack-up unit with different spudcan geometry might be mobilised. Gan (2009) investigated this effect by installing spudcans of different diameter nearby a consistently sized footprint (i.e. created by the same spudcan). The results were compared in terms of diameter ratio, which is the diameter of the spudcan creating the footprint (D_f) to the diameter of the reinstalling spudcan (D). For example, a diameter ratio D_f/D<1 refers to the case when the diameter of the spudcan adopted during footprint creation is smaller than the spudcan adopted during reinstallation. Figure 2.4 shows that within the range of critical offset distance (0.5 to 1.0D), the normalised peak horizontal force and the normalised peak moment generally increased with diameter ratio.

![Figure 2.4: The Effect of Diameter Ratio to the Jack-Up Reinstallation Response (after Gan 2009)](image)

2.2.4 Three-dimensional Effect of Footprint

The jack-up reinstallation problem could be simplified to a footing located to sloping ground composed of disturbed soil subjected to combined vertical, horizontal and moment loads. However, the three-dimensional nature of a footprint increases the complexity of the problem. Teh et al. (2006) attempted to assess the three-dimensional effect by conducting scaled model tests comparing the reinstallation of spudcan near a slope and near a footprint in sand. A 125mm diameter footing was considered in the test. The slope was about 36mm height and at an angle of about 30° (62mm wide). The
footprint was created by installation and extraction of spudcan and was about 75mm deep with a slope angle of about 32° (footprint radius of 125mm). Slightly different offset distances were used, with 0.5D and 0.4D used for the slope and footprint respectively. The vertical, horizontal and moment loads on the spudcan installed near the footprint were recorded and were all approximately three times larger than those near the slope. This indicates that three-dimensional effect of footprint could therefore be critical for the sand material. However, the actual contribution from the three-dimensional effect of footprint could not be determined because there were significant differences between the slope and the footprint geometry (the footprint being two times deeper and wider than the slope).

Understanding of the three-dimensional effect of the footprint would be useful for extrapolating the conventional bearing capacity and slope stability method for the prediction of jack-up reinstallation response. Therefore, further investigation on the three-dimension effect of a footprint is considered critical for the development of such a prediction method.

2.2.5 Footprint Geometry and Footing Reinstallation Response

The response of the reinstallation of a spudcan near a real footprint is influenced by changes of soil strength and the geometry of the footprint itself. Gan (2009) was the first to investigate the isolated effect of footprint geometry in the jack-up reinstallation. An idealised footprint crater was artificially created by burying a spudcan in an undisturbed soil sample at 2m depth (prototype scale), followed by consolidation in centrifuge facilities. After removing the spudcan, a footprint crater 1.0D in diameter and 0.2D deep was created. The soil strength was relatively undisturbed, allowing the footprint geometry to be investigated in isolation. Reinstallation test was then conducted at 100g. The spudcan was reinstalled at 0.5D offset from the footprint centre. Gan (2009) reported that the vertical forces were similar to those of the real footprint case. Although the trends are similar, surprisingly, both the peak horizontal force and moment were higher when compared to the real footprint case (Figure 2.5). A possible reason for the higher peak horizontal force and moment is that the comparison was conducted with a cylindrical-shaped footprint with a vertical wall, which resulted in a larger horizontal force and moment at a relatively shallow depth. The more realistic conical-shaped footprint, with a gentle sloping surface in soft and soft to firm clay should be considered instead. In addition, the influence of footprint geometry will also vary with the offset distance and therefore this should also be investigated.
2.3 SOIL STRENGTH

The soil strength under the footprint is controlled by the properties of the undisturbed soil and the operation procedures of the first jack-up installation and subsequent reinstallation. The soil failure mechanisms occurring during spudcan penetration, operational hold and extraction are extremely complex.

As described by Hossain (2008), and based on experimental PIV and large deformation finite element analyses, after the touchdown of the spudcan, the soil experiences a shallow heave failure mechanism and soil around the spudcan edge moves upward (Figure 2.6a). As penetration continues, a cavity is formed above the spudcan and the cavity stays open until critical penetration depth (H_s). H_s is a function of undrained shear strength profile of soil, soil density and spudcan diameter. As penetration continues below H_s, the soil starts to flow around the spudcan and the soil experiences a deep failure mechanism (Figure 2.6b). Purwana (2007) reported that the extent of lateral soil movement is within 1.3D laterally from the centre of installation, and the remoulded zone reduces to 0.75D as penetration continues to 0.75D below ground level.

As penetration continues and reaches the designed preload level, the bearing load is usually reduced to the working pressure level. The remoulded soil is consolidated as the excess pore pressure dissipates.

The failure mechanisms of spudcan extraction in normally-consolidated soil were presented in detail by Purwana (2007). First, the soil undergoes a breakout failure, separating the spudcan from the soil below it. After the breakout failure, the soil starts to flow from the top of the spudcan to the bottom. As the extraction continues, the soil above the spudcan, together with the spudcan, moves as a soil column (Figure 2.6c). For extraction near the seabed the soil slides toward the spudcan base as shown in

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Figure 2.5: Comparison of Recorded Loads Between a Idealised Footprint Crater and Real Footprint for a Spudcan Installed at an Offset of 0.75D (after Gan 2009)
Figure 2.6d. After complete extraction of the spudcan, the soil collapses into the cavity and a depression is formed near the ground surface.

The zone of soil influenced by the spudcan contains soil that has undergone shearing, been uplifted from below and finally collapsed to make a footprint. After extraction, the remoulded soil undergoes further consolidation as the excess pore pressure dissipates with time. All of this produces a complex pattern of soil strength under the footprint during the reinstallation. It varies spatially, with time, and will change with soil type and stress history.
2.3.1 Variation of Footprint Shear Strength

Cassidy et al. (2009) formed footprints in slightly over-consolidated soil and conducted t-bar characterisation tests to assess the footprint shear strength. Figure 2.7 shows the undrained shear strength profiles at different offset distances from the footprint centre. A reduction in undrained shear strength throughout $z_{FC}$ was reported. The footprint
shear strength was less than 40% of the original soil strength. The undrained shear strength was lowest at the footprint centre and then increased radially until 1.5D, at which point the footprint strength profile was similar to that of the undisturbed soil. Unfortunately, the operation time for forming the footprint and the elapsed time between footprint formation and t-bar characterisation tests were not published. However, as the t-bar penetrometer needed to replace the spudcan footing in the drum actuator, a procedure that takes over 10 mins, the prototype time is at least greater than 1.2 years. The pore-pressures in the soil are assumed to be fully dissipated. Therefore, the test results shows the long term undrained shear strength profiles.

![Shear Strength, $s_u$ (kPa)](image)

Figure 2.7: Comparisons of Undrained Shear Strength Profile at Different Offset Distance (Cassidy et al. 2009)

The operation time (OT) of the jack-up controls the consolidation time of the soil under the working pressure of the self-weight of the jack-up. Typical jack-up operation times range from a week for oil and gas exploration to 2 years for drilling production wells (Gan 2009). The elapsed time (ET) refers to the time gap between the extraction of a jack-up and the reinstallation and controls the consolidation time of the remoulded soil in the footprint. A typical elapsed time is around 3-5 years for mobilising a jack-up to an established site to drill additional wells or service existing wells (Gan 2009).
Gan (2009) performed a comprehensive investigation using centrifuge facilities to determine the effect of time on soil strength within a footprint. Footprints were created by installation and extraction of a spudcan in a clay soil sample. Ball penetrometer was adopted to evaluate the change of the undrained shear strength at 1, 3, 5 and 100 years prototype time after the footprint was created. The changes in undrained shear strength were compared using an undrained shear strength ratio $R_{su}$, which was defined as the undrained shear strength of the footprint to the strength of undisturbed soil. Gan (2009) compared footprints created in two different soil samples to allow comparison of the footprint undrained shear strength in normally-consolidated soil and in over-consolidated soil.

Figure 2.8 and Figure 2.9 show that for an operation time of 0, which is the spudcan being extracted immediately after reaching the design preload level, the undrained shear strength ratio increases radially up to 1.5D. In normally-consolidated clay, the undrained shear strength within the spudcan area (within 0.75D) is substantially remoulded. As the elapsed time increases to greater than 100 years, the soil around and below the spudcan area gains strength, and the undrained shear strength becomes higher than the undisturbed soil. For over-consolidated clay, the undrained shear strength ratio is similar to that for normally-consolidated clay, except that the variation with offset distance is less significant. The soil also gains strength as the elapsed time increases to greater than 100 years, but it only returns to the undrained shear strength of the undisturbed soil. For elapsed time greater than 100 years, a strong soil layer is also observed below $z_{FC}$ in both normally consolidated and over-consolidated clay.

As the operation time was increased to 2 years, there were no significant changes to the footprint soil strength in normally-consolidated soil, except that a strong soil layer was formed below the spudcan area even if the ET was relatively short (ET=1 year). Gan (2009) suggested that the soil underwent consolidation during the operation time, resulting in a higher $R_{su}$. Again, the soil around and below the spudcan area gained strength with time, and the undrained shear strength became higher than the undisturbed soil at an elapsed time greater than 100 years. For over-consolidated soil, the strength generally increased with the operation time; the trend for undrained shear strength changes was similar.

Gan (2009) concluded that for short operation and elapsed times, the variation of soil strength for both normally-consolidated and over-consolidated clays were similar. As operation time increases a strong soil layer is found at $z_{FC}$. The footprint soil strength increases with the elapsed time, and, in normally-consolidated soil, the footprint soil strength could be larger than the undisturbed soil strength.
Figure 2.8: Undrained Shear Strength Profile of Footprint in Normally-Consolidated Soil (after Gan 2009)
2.3.2 Strength of Undisturbed Soil

In addition to the comparison between reinstallation response in normally-consolidated soil and over-consolidated soil, Gan (2009) investigated the effect of undisturbed soil strength by conducting jack-up reinstallation tests in another four clay soil samples with different strengths. The test results are shown in Figure 2.10. The $s_u$ profiles of the soil samples are shown in Figure 2.11. Footprints were created by installing a spudcan into the soil samples until reaching a maximum preload pressure of 460kPa, followed by immediate extraction. The footprint geometry and $s_u$ of footprint are shown in Figure 2.11. The footprint geometries in the four soil samples were different due to the combined effect of different soil strength and different maximum penetration depth. After the soil sample achieved at least 90% degree of consolidation, the spudcan was reinstalled at 0.5D offset from the centreline of footprint. This simulated an infinitely long ET between first installation and reinstallation. In the firm soil and the first soft to firm soil ($s_u=38+3.5z$), distinct peak responses occurred at relatively shallow depth. The horizontal force and moment increased sharply from touchdown level and reached...
the maximum value between $z=1.5m$ to $2.5m$. The depth of peak responses coincided with the footprint toe level. Although the soil below this depth was highly disturbed, the horizontal force and moment reduced with further penetration. This indicates that the footprint geometry dominated the reinstallation response. In the second soft to firm ($s_u=30+3.5z$) soil sample, the horizontal force increased gradually with penetration depth and reached its maximum value at $z=9m$. The moment increased sharply until $z=1m$ to $2m$ then the gradient reduced and eventually reached a peak moment at approximately $11m$. It is due to the large variation of $s_u$ profile at around $11m$. Cassidy et al. (2009) also reported critical responses for deep reinstallation in soft soil. In the second soft and the softest soil, the horizontal force at around $11m$ depth even reduced to negative values, pushing the footing away from the footprint centre. In the softest soil, the negative peak horizontal force was larger than the peak horizontal force occurring during shallow depth reinstallation. This observation suggests that the variation of soil strength at greater depth is critical for the deep reinstallation in soft soil.
Figure 2.11: Geometry and Soil Strength of Footprints Created in Different Soil Samples
In summary, for reinstallation with long ET, the footprint geometry is critical to the reinstallation response in firm clay (with footprint created by relatively shallow maximum initial penetration depth) and the response at shallow depth in firm to soft soil. The soil heterogeneity effect is critical to deep reinstallation response in soft soil.

### 2.3.3 Effect of Operational Period and Elapsed Time on Jack-Up Reinstallation

Gan (2009) investigated the time effect by installing a spudcan at different elapsed times nearby a footprint formed after different operational times. An offset distance of 0.5D from the footprint centre was considered, and the results are presented in Figure 2.12 and Figure 2.13. Although the response was influenced by the combined effect of footprint geometry and the contoured changes in soil strength, the footprint geometry was similar between the tests, and the difference in the reinstallation responses could be assumed to be dominated by the change in footprint soil strength.

In the study of Gan (2009), in normally-consolidated clay, the horizontal force and the moment exhibited dual peak responses. The horizontal force and moment peaked at relatively shallow depth and again near $z_{FC}$ but in the opposite direction.

For reinstallation with long ET, the horizontal force and moment reduced shortly after reaching the peak values at relatively shallow depth ($z/z_{FC}=0.10$). It was because the remoulded soil around the footprint regained its strength with relatively long elapsed time. The reinstallation response was dominated by geometry rather than by soil heterogeneity. For the case of ET of 1 year, the strength of soil around the footprint was still highly variable and the shallow depth reinstallation response was influenced by the combined effect of footprint geometry and soil heterogeneity. Therefore, instead of exhibiting a peak value near the footprint toe level, the moment continued to increase with penetration depth until near $z/z_{FC}=0.6$.

The peak loadings during deep reinstallation near $z_{FC}$ were larger than at shallow depth. Gan (2009) explained that the footprint gained strength over time, and the strong soil layer at $z_{FC}$ pushed the spudcan away from the footprint centre inducing a negative horizontal force. The relatively high negative moment occurred near $z_{FC}$ and was due to the high variation of the undrained shear strength across the footprint (Figure 2.8). At $z_{FC}$, larger bearing resistance near the footprint centre tends to rotate the spudcan towards the footprint. The deep reinstallation response was also more insensitive to the change of OT compared to the shallow depth responses. However, the peak loadings near $z_{FC}$ generally increased with OT.
The reinstallation response in over-consolidated clay was similar to normally-consolidated clay, except that in over-consolidated clay the horizontal force and moment fluctuated around the peak values to a greater depth (near $z/z_{FC}=0.8$) before reducing to zero. This indicated that the soil heterogeneity has a greater effect on shallow depth reinstallation in over-consolidated clay.

Therefore, for both normally-consolidated soil and over-consolidated soil, the horizontal force and moment changed with OT and ET. The responses were influenced by the strong soil layer at $z_{FC}$. The extensive investigation by Gan (2009) provided a better understanding of the effect of soil strength on jack-up reinstallation.
2.4 JACK-UP STRUCTURAL PROPERTIES

Stewart and Finnie (2001) hypothesised that the horizontal load-displacement response of the spudcan is dependent on the jack-up structural properties, and illustrated this using Figure 2.14. Theoretically, an infinitely stiff leg will penetrate vertically as intended. On the other hand, a fully flexible jack-up leg will slide (and rotate if the argument of Stewart and Finnie (2001) is to be extended) into the footprint, as this is in the direction of least resistance. Therefore, a leg that is infinitely stiff would induce the maximum horizontal force and bending moment and a fully flexible leg would induce no force. Jack-up legs in reality have a certain amount of flexibility and tend to bend when reinstalled near the footprint (Foo et al. 2003).

![Figure 2.14: Prediction of Horizontal Load Displacement (after Stewart and Finnie 2001)](image)

Stewart and Finnie (2001)’s hypothesis was not substantiated with analytical solutions nor validated by experimental data. Therefore, further development is necessary for practical application. The main challenge in the development of the method was the incorporation the jack-up flexibility which is related to the structural properties of the entire jack-up unit and the foundation stiffness of any pre-embedded footings. This will control the magnitude of the induced forces and displacement paths of the jack-up leg. Furthermore, the method should also include the moment angular movement responses as the jack-up is likely to slide and rotate into the footprint during reinstallation.

2.4.1 Flexibility at the Leg-Hull Level

Due to the complexity of the problem and the large number of parameters involved in the jack-up reinstallation problem, a key simplification in most of the experimental investigations has been the modelling of the reinstalling jack-up unit as a single leg fully fixed to the actuator. This model is equivalent to a jack-up unit with zero lateral and
rotational flexibility at the hull level. The measured load should therefore be the upper bound. In an offshore scenario, the flexibility at the hull level is a function of structural properties of the jack-up unit and the fixity of any foundation embedded in soil. Gaudin et al. (2007) adopted a free-to-slide leg actuator connection, simulating a jack-up unit with infinite horizontal flexibility at the hull level. The moment gradually decreased to zero after reaching its peak value at approximately 0.25D below touch-down level, rather than increasing with penetration depth as in the case of a fully-fixed connection. The maximum moment is only half of the fully-fixed leg actuator test. This demonstrates that the finite flexibility at the hull level is critical in modelling jack-up reinstallation response. The results are shown on Figure 2.15.

![Figure 2.15: Moment Comparison Between a Free-to-Slide and Fully-Fixed Connection (after Gaudin et al. 2007)](image)

Bransby and Davison (2008) investigated the effect of foundation fixity on the bearing capacity of a foundation near a slope. Scaled model tests were conducted to investigate the bearing capacities of strip footings located adjacent to sand slopes. The strip footing was connected to the loading frame with three different fixity conditions 1) fully-fixed; 2) free-to-rotate and 3) free-to-slide and free-to-rotate (free) during penetration into the slope. It was reported that for the free-to-slide case the footing rotated towards the sloping ground. When the free connection was considered, the footing slid and rotated towards the sloping ground. The results are shown in Figure 2.16. For foundations located adjacent to 10° and 20° slopes, the bearing capacity reduced by 30% and 6% respectively when the foundation fixity changed from infinite to zero. However, the horizontal displacement of the footing adjacent to the 10° slope was only half of the case of 20°. The difference in angle of rotation was even more significant, the rotation
in the footing adjacent to the 10° slope was only 0.01 times of the case of 20°. Therefore the effect of foundation fixity is also related to the slope angle.

\[ \text{Figure 2.16: Change in Load-Displacement Response with Change in Foundation Fixity (after Bransby and Davison 2008)} \]

2.4.2 Jack-Up Leg Stiffness

As jack-up units are usually designed to operate in a range of water depths, the structural properties, which are dependent on the leg length, therefore vary with operation condition. It has been suggested that the bending stiffness of the leg would affect the jack-up reinstallation response (Stewart and Finnie 2001, Foo et al. 2003 and Dean and Serra 2004).

Jardine et al. (2002) developed a numerical model based on a Magellan class jack-up structure. The effect of jack-up leg stiffness was investigated by comparing the response of the realistic jack-up with a jack-up with leg stiffness that was 15 times larger or smaller. The horizontal forces and moment increased with increasing leg stiffness. Although there was a large reduction in rotation, the horizontal displacement decreased only slightly.

Cassidy et al. (2009) compared the reinstallation responses of four model jack-up legs with bending stiffness \( (EI/L^3) \) ranging from 1.60 to 25.7MN/m. Surprisingly, the penetration responses were similar among the four model jack-up legs, with almost the same peak horizontal force and moment measured for each. Gan (2009) conducted a similar comparison and considered two model jack-up legs with a bending stiffness of
3.60 and 55.1MN/m. Although there was a 20% increase in the maximum moment when the stiffer leg was adopted, there was no significant difference in the maximum horizontal force. The comparisons had already covered a wide range of realistic jack-up leg stiffness values. However, because the model jack-up leg was fully fixed to the actuator, the rest of the jack-up unit was modelled as an infinitely stiff structure. The jack-up reinstallation response might be controlled by the structural stiffness of the entire jack-up unit, rather than by the single leg stiffness.

The experimental results contributed to a better understanding of this complex problem. However, the measured loads are potentially significantly lower and the displacement in the jack-up should be higher if the system behaviour is taken into account. The investigation of full three-legged jack-up responses was recommended.

2.5 CURRENT DESIGN GUIDELINE AND MITIGATION MEASURES

SNAME (2002) provides general advice on two operational sequences of jack-up reinstallation. First, if an identical jack-up unit is mobilised, the jack-up should be repositioned in exactly the same footprint position to avoid loading spudcans laterally and eccentrically. Foo et al. (2003) reported that in such a case the horizontal force and the moment acting in the jack-up leg were similar to penetration in undisturbed soil without excessive lateral spudcan displacement. However, it is unlikely that the same jack-up unit or a jack-up unit with an identical design is mobilised for reinstallation. It is therefore recommended to carefully position the jack-up with one footing over a footprint and the others in undisturbed soil. In both cases, reliable records of the exact location and depth of footprints are recommended. SNAME (2002) provides a general guideline for keeping a minimum safe offset distance of one diameter between the footprint and the reinstalling spudcan. When it is impossible to avoid spudcan-footprint interactions, infilling the footprints with imported materials is recommended. However, no guideline is given on the type and properties of infilling material.

In addition to infilling the footprint, other possible mitigation measures including stomping, monitoring rack phase difference and using skirted footing, have been proposed in recent publications and are discussed in detail in subsequent sections.

Dean and Serra (2004) also presented some possible mitigation measures, mainly based on the alteration of the structural components of the jack-up unit. However, these options are still in the early conceptual stage, and further assessment is needed to confirm their practicality. These options will therefore not be discussed further in this study.
2.5.1 Infilling the Footprint Crater

Jardine et al. (2002) established a numerical model to investigate the effectiveness of infilling a footprint. A plane strain finite element model was adopted and the jack-up unit was simplified to a single jack-up leg fully fixed at the leg-hull level (Figure 2.17). The footprint was modelled as a cavity with a vertical wall, and the footprint undrained shear strength was 93% of that of undisturbed soil. The diameter of the footprint was assumed to be the same as the diameter of the reinstallation spudcan. Two infill materials, namely medium dense sand (unit weight=8kN/m³ and friction angle=33º) and dense gravel (unit weight=10kN/m³ and friction angle=40º), were considered. A series of wished-in-place models were analyzed to determine the load and displacement in the jack-up leg at different depths of reinstallation. It was found that the horizontal force and moment were both lower when gravel was adopted instead of sand. A larger difference in stiffness between the infilled material and the seabed material would improve the effectiveness of this mitigation measure. However, in both cases, the maximum horizontal force and moment exceeded the structural capacity of the jack-up leg.

<table>
<thead>
<tr>
<th>Layer 1</th>
<th>Firm to stiff clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer II</td>
<td>Very stiff to hard clay</td>
</tr>
<tr>
<td>Layer III</td>
<td>Sand</td>
</tr>
</tbody>
</table>

Figure 2.17: Modelling of Jack-up Reinstallation Near Infilled Footprint by the Plane-Strain Finite Element Method (after Jardine et al. 2002)
The investigation was extended by Grammatikopoulou et al. (2007). A similar numerical model was established, investigating jack-up the reinstallation in North Everest, North Sea. Footprints in clay inter-beded with sand layers were considered in the analysis. A more realistic footprint geometry with sloping ground was modelled, and the maximum footprint depth was 3m. Two practical infilling options were considered. The first option was infilling the footprint with gravel. In the second option, the infilled footprint was capped with a gravel platform. Although both options reduced leg forces, the horizontal force and moment still exceeded the structural capacity of the jack-up leg. Based on the analyses by Jardine et al. (2002) and Grammatikopoulou et al. (2007), infilling the footprint might not be an effective solution to mitigate the problem.

In theory, infilling the crater could eliminate the influence of the footprint geometry. However, as discussed in Section 2.3, the peak horizontal force and moment at greater depth (near \( z_{FC} \)) is controlled by the soil strength variation which cannot be eliminated by the infilling method. Alternatively, the footprint can be infilled with very stiff material (e.g. gravel) so that the spudcan cannot penetrate through the infilled material. This could eliminate the effect of soil strength variation below the footprint. However, this creates a punch through failure hazard scenario.

### 2.5.2 Stomping

Jardine et al. (2002) reviewed the application of stomping as a mitigation measure to the jack-up reinstallation problem. Stomping involves raising and lowering the jack-up leg over and away from the footprint, in order to displace the soil towards the footprint (Figure 2.18).

![Figure 2.18: Illustration of Stomping (after Jardine et al. 2002)](image)

Similar to infilling the crater, stomping could eliminate the footprint geometric effect and would have no effect to the soil strength variation which control the peak responses at \( z_{FC} \). Stomping is also subject to practical limitations. This method is not applicable if
the clearance between the footprints and the fixed platform is limited because the jack-up unit cannot be mobilised close to the footprint. The process of stomping could also involve significant rig-time and is also constrained by the weather conditions. Furthermore, there was no documented numerical or experimental investigation to confirm the effectiveness of stomping.

2.5.3 Rack Phase Difference (RPD)

Rack Phase Difference (RPD) refers to the difference in elevation between jack-up legs. As shown in Figure 2.19, RPD can be used to measure the inclination of the leg relative to the hull and hence may be used to estimate leg loads. Foo et al. (2003) suggested using real-time RPD monitoring during jack-up reinstallation to prevent leg bending failure and to monitor sliding of the spudcan. If within limit, the RPD can be corrected through careful jacking of the other legs. The loading acting in the leg located nearby the footprint can therefore be transferred to the other legs. This allows mobilisation of the stiffness of the entire jack-up and therefore resisting the load in a more efficient way.

![Figure 2.19: Schematic View of RPD Effect (after Stonor et al. 2004)](image)

RPD (face AB) = ∆A + ∆B
RPD (face AC) = ∆A + ∆C
RPD (face BC) = ∆B + ∆C

If chord B moves upwards relative to chord A, brace A1-B2 goes into tension and brace B1-A2 goes into compression.
2.5.4 Skirted Footing

Teh et al. (2006) investigated the responses of a skirted footing installed near a slope. Scaled model tests under unit gravity were conducted in heavily over-consolidated clay. Teh et al. (2006) reported that although the moments were comparable for a skirted footing versus a non-skirted footing, the skirted footing resulted in larger horizontal force but in the opposite direction, pushing the footing away from the slope toe. There was no significant difference in moment for the skirted footing when compared to a spudcan without a skirt. This result is actually in good agreement with the numerical investigation conducted by Carrington et al. (2003). From the large strain three-dimensional finite element analysis, installation of a skirted spudcan resulted in a larger horizontal force, but in the opposite direction, when compared to the standard spudcan case.

Gaudin et al. (2007) also investigated the effectiveness of using a skirted footing as a mitigation measure. A free-to-slide model jack-up leg actuator was adopted, and the horizontal displacement was reduced by 27% compared to the fully-fixed model without significant change in the maximum moment. The results were significantly different from those conducted with a fully-fixed leg (Teh et al. 2006, Carrington et al. 2003). Therefore, the assumption of the model leg-actuator connection is critical for assessing the effectiveness of using a skirted footing as a mitigation measure.

2.6 SUMMARY AND RESEARCH OF THIS THESIS

2.6.1 Summary

Although the jack-up reinstallation problem is a clearly known hazard, only limited publications are available addressing this issue. The problem is complex and involves a large number of parameters. Previous studies mainly adopted physical modelling to investigate the response jack-up reinstallation.

In most of the physical modelling conducted thus far, a real footprint with highly varying soil strength was first generated, followed by spudcan reinstallation. Publications presented detailed observations and precise measurements of the footprint geometry (e.g. Cassidy et al. 2009, Gan 2009). However, the role of the footprint geometry in jack-up reinstallation is still not understood, because, apart for a single test of Gan (2009), it has not been studied in isolation. All of the studies presenting responses of reinstallation near a footprint were subjected to the combined effect of footprint geometry and changing soil strength profile.

Gan (2009) presented a comprehensive investigation on the footprint soil properties and their governing factors. The soil strength varied radially from the footprint centre and from the ground surface. It is evident that the soil around the footprint gains strength
with operation time and elapsed time. The peak horizontal force and the moment are both, therefore, also affected by time. The strong soil layer formed beneath $z_{FC}$ has a significant effect on the direction and the peak horizontal force during jack-up reinstallation at greater depth.

A number of researchers discussed the importance of jack-up structural properties in relation to the problem (Stewart and Finnie 2001, Gaudin et al. 2007, Gan 2009, Cassidy et al. 2009), however, these properties have never been investigated. Due to the restrictions in testing apparatuses, the jack-up units have been simplified as a single jack-up leg connected to the actuator. Most of the studies adopted a fully-fixed leg actuator connection, modelling a jack-up with zero flexibility at the leg-hull level. The infinitely low flexibility at the jack-up leg actuator connection might have overshadowed any effect of the variation of the model jack-up leg bending stiffness. Gaudin et al. (2007) investigated the upper bound jack-up flexibility by developing and using a free-to-slide leg-actuator connection. Substantial changes to the moment were noted, which suggested that the assumptions about the leg actuator connection are critical to the jack-up reinstallation response. No investigation of anything in between has been conducted.

Herein, a brief review of mitigation methods was presented. Numerical analysis indicated that infilling footprint might not be a effective solution to mitigate the problem (Jardine et al. 2002, Grammatikopoulou et al. 2007). Gaudin et al. (2007) presented encouraging results on the use of skirted footing, which deserves further investigation.

### 2.6.2 Research of This Thesis

From the literature review presented above, the following key areas were identified for detailed research in this thesis:

1. Due to the high complexity of the jack-up reinstallation response, a systematic investigation on the isolation effect of the three governing parameters: footprint geometry, soil heterogeneity and jack-up structural properties is necessary to improve the understanding of the problem;

2. A detailed investigation has been presented by Gan (2009) on the soil heterogeneity around the footprint and its effect to jack-up reinstallation. Therefore, this study aims to fill the knowledge gap on the role of footprint geometry and jack-up structural properties to the jack-up reinstallation response.

3. The experimental focus of this study in relation to the achievements and limitations of the other published experimental studies is summarised in terms of the three identified key areas in Table 2.4.
Chapter 2: Literature Review

4. The model jack-up leg is usually modelled with zero flexibility because it is attached to the actuator with a fully-fixed connection. Therefore, a new testing method will be developed to model a jack-up unit with specific structural properties during a reinstallation;

5. The prediction of loading and movement on the jack-up during reinstallation would be beneficial to the industry. This research will therefore develop a simple prediction method for the assessment of reinstallation response for a selection of jack-up units with sufficient structural capacity;

6. Although being perceived as a possible mitigation method for the reinstallation problem, the reinstallation response of skirted footing are not well understood. This research will conduct further investigation focussing on the failure mechanism of a skirted footing during reinstallation.
## Table 2.4: Findings on the Three Key Parameters and the Focus of This Study

<table>
<thead>
<tr>
<th>References</th>
<th>Footprint Geometry</th>
<th>Real Footprint</th>
<th>Structural Stiffness</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Slope</td>
<td>Footprint Cavity</td>
<td>Soil Strength Measured</td>
<td>Soil Strength Not Measured</td>
</tr>
<tr>
<td>Stewart and Finnie (2001)</td>
<td>-</td>
<td>-</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>Teh et al. (2006)</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>Gaudin et al. (2007)</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>Cassidy et al. (2009)</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>Gan (2009)</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>This Study</td>
<td>Chapter 3 Chapter 4</td>
<td>Chapter 3 Chapter 7 Chapter 9</td>
<td>-</td>
<td>Chapter 4 Chapter 8 Chapter 9</td>
</tr>
</tbody>
</table>
CHAPTER 3. GEOMETRIC EFFECT OF FOOTPRINT

3.1 INTRODUCTION

The response of a jack-up during reinstallation is dependent on the footprint geometry, soil strength heterogeneity within the footprint and the structural properties of the jack-up unit. A study of literature has revealed that for reinstallation with long elapsed time, the peak loading occurred at relatively shallow depth and the reinstallation response was dominated by the footprint geometry (refer to Chapter 2 for details). However, the isolated effect of footprint geometry is still unknown. An understanding of the influence of each of these parameters in isolation is necessary to identify the critical parameter to the reinstallation response. This is one of the key objectives of this thesis.

Physical modelling using the UWA geotechnical centrifuge facilities has been conducted to assess the complex foundation response in the problem. Physical modelling allows data to be collected on a replicated event for the refinement of numerical modelling and formulation of empirical relations. The major advantage of centrifuge modelling is it allows realistic assessment of foundation response with the stress level in the soil comparable to larger prototype conditions offshore. Parametric studies can also be carried out in a controlled environment in a cost-effective manner.

In this chapter, the geometric effect of the footprint has been isolated and investigated in details. The reinstallation test was conducted by installing a model jack-up leg and spudcan near a manually cut footprint into the clay surface. This artificially cut footprint was conical in shape and represented an idealised footprint cavity. The model jack-up leg was rigidly connected to the centrifuge actuator and therefore represented a fully-fixed jack-up leg with zero flexibility at the jack-up leg hull level. The idealised footprint cavity modelled a footprint with consistent geometry and created a homogeneous strength profile eliminating soil heterogeneity from the physical model. Furthermore, the fully-fixed model jack-up leg eliminated the effect of structural stiffness. Therefore, the only variable studied in the tests reported in this chapter is the footprint geometry itself. The experimental motivation of this chapter is shown in the schematic diagram in Figure 3.1 and the idealised assumptions in Figure 3.2.
In this chapter a brief overview on the centrifuge modelling principles is presented, followed by a detailed description on the experimental apparatus, testing procedures and programme. The experimental results of tests conducted nearby idealised footprint cavities of different depth, diameter and slope angle are reported. Finally, discussion on reinstallation response and a quantitative comparison with other publications are presented.
3.2 PRINCIPLES OF CENTRIFUGE TESTING

A comprehensive overview on the principle of centrifuge modelling can be found in Taylor (1995). A soil model placed at the end of a centrifuge arm can be accelerated to a radial acceleration at a scale N times the earth’s gravity. The body force is increased and the model can simulate the stress level at field, hence the in-situ strength and stiffness of soil. The vertical stress of a soil particle at a particular depth \( z_{\text{model}} \) in the centrifuge model will be identical to that of in the corresponding prototype depth \( z_{\text{prototype}} = N \times z_{\text{model}} \). Based on the above basic scaling factor for stress, together with dimensional analysis, scaling factors have been developed for various dimensions. Table 3.1 shows the scaling factors relevant to this study.

Table 3.1: The Scaling Factors

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Relationship (model/prototype)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravity</td>
<td>( N )</td>
</tr>
<tr>
<td>Stress</td>
<td>1</td>
</tr>
<tr>
<td>Strain</td>
<td>1</td>
</tr>
<tr>
<td>Length</td>
<td>( \frac{1}{N} )</td>
</tr>
<tr>
<td>Force</td>
<td>( \frac{1}{N^2} )</td>
</tr>
<tr>
<td>Moment</td>
<td>( \frac{1}{N^3} )</td>
</tr>
<tr>
<td>Density</td>
<td>1</td>
</tr>
<tr>
<td>Mass</td>
<td>( \frac{1}{N^3} )</td>
</tr>
<tr>
<td>Time (consolidation)</td>
<td>( \frac{1}{N^2} )</td>
</tr>
</tbody>
</table>

3.3 EXPERIMENTAL APPARATUS

3.3.1 Drum Centrifuge Facility at UWA

The drum centrifuge at the University of Western Australia (UWA) was established in 1999 (Figure 3.3). The drum has a diameter of 1.2m, channel depth of 0.2m and channel height of 0.3m. The centrifuge can create an effective gravity of 485g through a rotation of 850rpm. The centrifuge has two concentric shafts, one to drive the sample channel, and one to drive a central tool table actuator. A clutch allows both shafts to rotate at the same speed or independent movement can be provided if it is declutched. The central tool table actuator can also move vertically and radially, in all allowing movements in three-degrees-of-freedom. The maximum loading rate for the two axes is 4.75mm/s. There are two onboard data acquisition systems, one within the channel and
one on the control tool table. The data are transferred to the control room through a slip-ring system. A full description of the drum centrifuge apparatus can be found in Stewart et al. (1998).

A major advantage of the drum centrifuge is that tools can be changed without stopping the channel. This avoids any need to reconsolidate the soil sample after the centrifuge has stopped. Furthermore, the drum channel provides a relatively large scaled plan area allowing a number of tests to be carried out in the same soil sample. These factors were particularly useful for the relatively large number of tests required for the parametric study on the geometric effect of footprint, as described in this thesis.

Figure 3.3: Drum Centrifuge Facility at UWA

### 3.3.2 Penetrometers

A t-bar penetrometer (t-bar) was used in this study to assess the undrained shear strength profile of the soil sample. The t-bar was initially developed by Stewart and Randolph (1991) for characterising centrifuge soil samples. It is increasingly used in the offshore environment to estimate the undrained shear strength profile of seabed (Low et al. 2009, DeGroot et al. 2010). The significant advantage of a t-bar compared to a conventional cone is that soil can flow completely around the t-bar cylinder. This allows interpretation using rigorous plasticity solutions to link the bearing pressure to the soil strength.

The t-bar used in the drum centrifuge consists of a cylindrical bar, 5mm in diameter and 20mm in length, attached perpendicularly to the end of a shaft (Figure 3.4). Strain gauges are located immediately above the cylindrical bar to record the net bearing pressure ($q_{t-bar}$) required to continuously penetrate the t-bar.
Chapter 3: Geometric Effect of Footprint

Figure 3.4: T-bar Penetrometer Used in Testing

The measured bearing pressure on the t-bar \( (q_{t-bar}) \) is converted to the undrained shear strength of soil according to Equation (3.1).

\[
s_u = \frac{q_{t-bar}}{N_{t-bar}} \tag{3.1}
\]

Where \( q_{t-bar} \) is the corrected t-bar penetration resistance, \( N_{t-bar} \) is the resistance factor and \( s_u \) is the undrained shear strength. The bearing factor \( N_{t-bar} \) increases non-linearly from the surface until a deep penetration mechanism is established (Figure 3.5). The variation of \( N_{t-bar} \) at shallow depth governs the soil strength in the first 3-4 diameters of penetration depth. This depth is critical to this study as it coincides with the footprint depth. Therefore, two different \( N_{t-bar} \) namely \( N_{t-deep} \) and \( N_{t-shallow} \) were evaluated.

Figure 3.5: Different Failure Mechanisms for Shallow and Deep T-bar Penetration (after White et al. 2010)
Randolph and Houlsby (1984) determined the $N_{t\text{-bar}}$ for deep flow-round failure mechanism using rigorous plasticity solutions. Values ranged from 9.14 for a fully smooth interface to 11.94 for a fully rough interface. Since the t-bar used in the centrifuge is neither fully smooth, nor fully rough, a $N_{t\text{-deep}}$ of 10.5 was adopted, as suggested by Randolph and Houlsby (1984), Stewart and Randolph (1991) and Watson (1999). This value had been confirmed through experimental calibrations using a range of material types, stress histories and stress levels by Watson (1999).

The penetration resistance at shallow embedment is affected by the surface heave rather than deep flow-around failure mechanism, as well as the soil buoyancy. White et al. (2010) presented a methodology for the determination of shallow depth bearing factor $N_{t\text{-shallow}}$ based on theoretical considerations and large deformation finite element analyses and suggested:

$$N_{t\text{-shallow}} = 2 + (N_{t\text{-deep}} - 2)\left(\frac{\bar{w}}{w_{\text{deep}}}\right)^p$$

$$\bar{w}_{\text{deep}} = 2.58\left(\frac{S_u}{\gamma' D}\right)^{0.46} + 0.24\left(\frac{S_u}{\gamma' D}\right)^{-0.63}$$

$$p = 0.61\left(\frac{S_u}{\gamma' D}\right)^{-0.31}$$

where $\bar{w}$ = the penetration depth of t-bar invert normalised by the diameter of t-bar.

A miniature ball penetrometer developed by Lee et al. (2011) was used to evaluate the variation of $S_u$ across the footprint (Figure 3.6). The miniature ball penetrometer was made of epoxy material instead of the conventional aluminium alloy used in the t-bar. This allows the strain gauges to be embedded inside the shaft, minimising the diameter of the shaft and hence the diameter of the spherical probe attached at the end. The diameter of spherical probe was only 5mm with a 2.5mm diameter shaft. A smaller test site was required for strength profiling using the miniature ball penetrometer compared to the t-bar penetrometer. This allowed more $S_u$ profiling tests to be carried out at closer space within the idealised footprint cavity. As recommended in Lee et al. (2011), a resistance factor, $N_{\text{ball}}$ of 13.5 was adopted to evaluate the undrained shear strength, which is slightly higher than that recommended by Watson (1999). Lee et al. (2011) suggested that the higher $N_{\text{ball}}$ was related to the higher interface friction ratio of the epoxy and clay soil compared to aluminium and clay. Lee et al. (2011) and Gan (2009) obtained satisfactory results from using the ball penetrometer in over-consolidated clay soil and dense sand overlaying clay soil.
3.3.3 Model Footings

Three model footings were used in the experiments. Two were a model flat-base circular footings and the other was a skirted footing. All were made of solid aluminium and their dimensions are presented in Table 3.2 and Figure 3.7. The flat-base footings were used to eliminate the variables related to spudcan geometry, including the apex angle, central apike and cone angle. The touchdown level of the flat-base footing could also be identified clearly. The flat-base footings were fabricated with a curvature matching the curved surface of the clay sample in the drum channel (Figure 3.7) to ensure full contact with the clay sample immediately at touchdown. For a direct comparison, a skirted footing of the same diameter was also considered. The skirted footing was fabricated with a 3mm diameter drainage hole in the top plate to replicate the drainage mechanism in a prototype skirted footing. The footings were connected to the model jack-up leg through a screw connection. This achieved a rigid connection.

Table 3.2: Prototype and Model Properties of Footings

<table>
<thead>
<tr>
<th>Foundation Type</th>
<th>Diameter_{model} (mm)</th>
<th>Depth_{model} (mm)</th>
<th>Prototype Diameter (m)</th>
<th>Prototype Thickness/Skirt Length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Footing</td>
<td>60</td>
<td>6.39</td>
<td>15</td>
<td>1.725</td>
</tr>
<tr>
<td>Footing .80 (Control Test only)</td>
<td>80</td>
<td>7.09</td>
<td>15</td>
<td>1.33</td>
</tr>
<tr>
<td>Skirted Footing</td>
<td>60</td>
<td>20</td>
<td>15</td>
<td>5</td>
</tr>
</tbody>
</table>
3.3.4 Model Jack-Up Leg

The jack-up reinstallation problem was simplified and modelled by reinstallation of a single jack-up leg nearby a footprint. The model jack-up leg was connected to the actuator through a screw-in connection piece, giving a fully-fixed connection. This simulated a model jack-up with zero flexibility at the leg hull level.

The model jack-up leg was fabricated by the Civil Engineering Workshop of UWA for the centrifuge tests. The model jack-up leg was a scaled version of KeppelFELS Mod “V” A Class jack-up unit (Table 3.3). However, the relatively complex truss configuration of the prototype jack-up leg was replaced by a thin wall hollow aluminium cylinder model with the same flexural stiffness (Figure 3.8). The axial stiffness could not be modelled, as a very large diameter and very thin wall tube would be required to model the relatively low axial stiffness. This might result in buckling failure of the model jack-up leg. However, the axial deformation is usually not a concern in jack-up installation.
Table 3.3: Prototype and Model Properties of Jack-up Leg

<table>
<thead>
<tr>
<th>Properties</th>
<th>Prototype</th>
<th>Scaling Factor</th>
<th>Model @ 250g</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cross-sectional Area</td>
<td>-</td>
<td>-</td>
<td>6.72</td>
</tr>
<tr>
<td>Outer Dia. (mm)</td>
<td>-</td>
<td>-</td>
<td>24.3</td>
</tr>
<tr>
<td>Thickness (mm)</td>
<td>-</td>
<td>-</td>
<td>1.5</td>
</tr>
<tr>
<td>L (mm)</td>
<td>47500</td>
<td>N</td>
<td>190</td>
</tr>
<tr>
<td>EI (kNmm²)</td>
<td>2.76E+15</td>
<td>N²</td>
<td>7.07E+5</td>
</tr>
<tr>
<td>EI/L³ (kN/mm)</td>
<td>25.75</td>
<td>N</td>
<td>0.10</td>
</tr>
</tbody>
</table>

Axial and bending strain gauges were attached to the model jack-up leg to measure the axial strain and bending strain (Figure 3.8). While the axial gauges allow the direct measurement of vertical force (V) acting on the jack-up leg, the moment (M) and the horizontal force (H) at the load reference point (LRP) were evaluated using statics from the bending gauges along the jack-up leg. The sign convention and the terminologies were standardised as shown in Figure 3.9.
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Figure 3.8: K-lattice Jack-up Leg (Courtesy of Keppel) and Model Jack-up Leg Used in Test (All Dimensions in mm)

Figure 3.9: Sign Convention and Definition of Terminology

\[ s_u(z) = s_{um} + k z \]
Figure 3.10 shows the arrangement of the model jack-up leg in the drum centrifuge. Gaudin et al. (2007) and Cassidy et al. (2009) reported that the response of a fully-fixed jack-up leg was relatively insensitive to the stiffness of jack-up leg. Therefore, only one jack-up leg was adopted in this study.

![Figure 3.10: A Fully-Fixed Connection between the Model Jack-up Leg and the Actuator in Drum Centrifuge](image)

### 3.3.5 Preparation of Soil Sample

An over-consolidated soil profile was targeted in the testing as it both represents many regions of hydrocarbon exploration and is a profile where deeper footprints can be created (due to higher surface strength than normally-consolidated soil). Kaolin clay soil that has been used extensively in UWA was adopted in this test and the key properties are presented in Table 3.4.

The dry kaolin powder was mixed with water to form slurry with a water content of 120%. This is approximately twice the liquid limit of the clay. The kaolin slurry was then placed in the drum channel and consolidated overnight at 300g to create a normally-consolidated kaolin clay sample. The drum channel was stopped and a layer of geotextile was placed on top of the clay sample. Sand was then sprayed onto the geotextile until a 35mm surcharge layer was formed. The sample was further consolidated at 300g. The changes in pore pressure were monitored by the readings from the pore pressure transducers located in the soil sample. After achieving stable pore-pressures after 2 days, the drum channel was then stopped and the geotextile and the surcharge layer were carefully removed, leaving an over-consolidated clay sample of approximately 130mm depth. The over-consolidation was further enhanced by
performing tests at 250g. Photographs of the preparation process are provided in Figure 3.11.

Table 3.4: Properties of Kaolin Clay (after Stewart 1992 and Acosta-Martínez 2008)

<table>
<thead>
<tr>
<th>Properties of Kaolin Clay</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid Limit</td>
<td>LL</td>
<td>61%</td>
</tr>
<tr>
<td>Plastic Limit</td>
<td>PL</td>
<td>27%</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>Ip</td>
<td>34%</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>G_s</td>
<td>2.6</td>
</tr>
<tr>
<td>Angle of Internal Friction</td>
<td>$\phi'$</td>
<td>23°</td>
</tr>
<tr>
<td>Submerged Unit Weight</td>
<td>$\gamma'$</td>
<td>6.82 kN/m$^3$</td>
</tr>
<tr>
<td>Critical State Frictional Constant</td>
<td>M</td>
<td>0.92</td>
</tr>
<tr>
<td>Slope of Normal Consolidation Line</td>
<td>$\lambda$</td>
<td>0.207</td>
</tr>
<tr>
<td>Slope of Swelling Line</td>
<td>K</td>
<td>0.044</td>
</tr>
<tr>
<td>Coefficient of Consolidation (at 20kPa)</td>
<td>$c_v$</td>
<td>2 m$^2$/year</td>
</tr>
</tbody>
</table>

Figure 3.11: Preparation of Over-consolidated Clay Sample
3.3.6 Preparation of Idealised Footprint Cavity

The geometry of a footprint is dependent on a number of parameters, including the footing geometry, foundation loading and soil properties (SNAME 2002, Cassidy et al. 2009). In order to conduct a systematic parametric study, the geometry of the footprint was simplified into an idealised conical shape with diameter and depth related to the footing diameter. It was important to ensure that the geometry of the idealised footprint cavities considered was within practical range. Published data on the geometry of a real footprint (footprint formed by penetration and extract of a jack-up spudcan) was reviewed and a summary is presented in Table 3.5. Unfortunately only one publication was found on field measurement of footprint geometry, and that was on very stiff clay. Therefore, the review includes the footprint geometries formed in centrifuge testing. The maximum penetration depth during footprint creation, \( z_{FC} \), ranged from 0.11D to 0.57D. The diameter of footprint formed in slightly over-consolidated soil was approximately two times the footing diameter, while the footprint depth varied from 0.11D to 0.28D.

**Table 3.5: Summary of Published Data of Footprint Geometry in Clays**

<table>
<thead>
<tr>
<th>Publications</th>
<th>Soil Strength</th>
<th>( q_{\max} )</th>
<th>( z_{FC} )</th>
<th>D</th>
<th>( z_F )</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stewart and Finnie (2001)</td>
<td>( s_u = 12+2.65z ) kPa</td>
<td>327kPa</td>
<td>0.57D</td>
<td>2.1D</td>
<td>0.21D</td>
<td>Centrifuge Testing</td>
</tr>
<tr>
<td>Cassidy et al. (2009)</td>
<td>( s_u = 7.5+2z ) kPa</td>
<td>152kPa, 228kPa, 304kPa</td>
<td>0.16D, 0.31D, 0.46D</td>
<td>2.0D</td>
<td>0.11D, 0.12D, 0.12D</td>
<td>Centrifuge Testing</td>
</tr>
<tr>
<td>Gan et al. (2009)</td>
<td>( s_u = 50+4.1z ) kPa</td>
<td>460kPa</td>
<td>0.2D</td>
<td>1.0D</td>
<td>0.21D</td>
<td>Centrifuge Testing</td>
</tr>
<tr>
<td></td>
<td>( s_u = 30+3.8z ) kPa</td>
<td></td>
<td>0.5D</td>
<td>1.6D</td>
<td>0.23D</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( s_u = 28+1.65z ) kPa</td>
<td></td>
<td>1.3D</td>
<td>2.0D</td>
<td>0.2D</td>
<td></td>
</tr>
<tr>
<td>Jardine et al. (2002)</td>
<td>( s_u = 40+6z ) kPa</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>0.28D</td>
<td>Field Measurement</td>
</tr>
</tbody>
</table>

Based on the review, three basic footprints were developed in this study. The footprint diameter was kept at two times the footing diameter, with depths ranging from 0.17D to 0.67D. These three basic footprints, namely TA, TB and TC, were modified for parametric study on the effect of footprint diameter (\( x_F \)) and depth (\( z_F \)). The three-dimensional effect of the footprint was also investigation by considering a sloping ground in the same angle as TB. It has been named SB and detailed discussion is presented in Section 3.4.3. The full details of the footprint geometries are presented in Table 3.6.
Table 3.6: Dimension of Footprint Type

<table>
<thead>
<tr>
<th>Footprint Type</th>
<th>Model Dimension</th>
<th>Prototype Dimension</th>
<th>η (°)</th>
<th>Normalised Dimension</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>x_F (mm)</td>
<td>z_F (mm)</td>
<td>x_F (m)</td>
<td>z_F (m)</td>
</tr>
<tr>
<td><strong>TA</strong></td>
<td>10</td>
<td>2.50</td>
<td>2.50</td>
<td>9.50</td>
</tr>
<tr>
<td><strong>TB</strong></td>
<td>120</td>
<td>20</td>
<td>30</td>
<td>5.00</td>
</tr>
<tr>
<td><strong>TC</strong></td>
<td>40</td>
<td>10.00</td>
<td>10.00</td>
<td>33.70</td>
</tr>
<tr>
<td><strong>TA-3D</strong></td>
<td>180</td>
<td>15</td>
<td>45</td>
<td>3.75</td>
</tr>
<tr>
<td><strong>TB-1D</strong></td>
<td>60</td>
<td>10</td>
<td>15</td>
<td>2.50</td>
</tr>
<tr>
<td><strong>TB-13D</strong></td>
<td>80</td>
<td>13</td>
<td>20</td>
<td>3.33</td>
</tr>
<tr>
<td><strong>TB-2D</strong></td>
<td>160</td>
<td>26.67</td>
<td>30</td>
<td>5.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>SB</strong></td>
<td>120</td>
<td>20</td>
<td>30</td>
<td>5.00</td>
</tr>
</tbody>
</table>

For control test @187.5g
Three separate “V-shape” blades were fabricated for the creation of the three different idealised footprint cavities. All footprints were created with the centrifuge at rest, as shown in Figure 3.12. The blade was firstly mounted to the drum channel and securely positioned at the proposed test site. The blade was then rotated to remove soil and to form a cavity of ideal conical shape. This continued until the designed footprint depth was reached. Using this method minimises disturbance of the soil underneath the idealised footprint cavity. This will be confirmed in Section 3.5.2, when t-bar characterisation test results on the idealised footprint cavity are interpreted. The geometry of the idealised footprint cavities was confirmed by laser profiling. An example of a laser profile result is shown in Figure 3.13.
Chapter 3: Geometric Effect of Footprint

3.4 EXPERIMENTAL PROCEDURES AND PROGRAMME

3.4.1 Testing Procedures

After reaching the designed testing acceleration level of 250g, the model jack-up leg was penetrated at an offset distance away from the idealised footprint cavity until reaching a penetration depth of 1.5D below the soil surface. This is referred to as a footing reinstallation test. The vertical, horizontal and moment load were measured throughout the test. All tests were carried out with free water above the soil sample to simulate the effective unit weight of the clay soil in the offshore environment. A footing installation test was also conducted by penetrating and extracting the jack-up leg into the flat soil surface (away from any footprint) to create a real footprint and for comparison with reinstallation test results. To distinguish this test from those in a footprint, this test is named “Flat Surface” (FS). The real footprint formed in FS was then examined and laser profiling was conducted to validate the footprint geometries presented in Section 3.3.6. The undrained shear strength profile of the soil sample and the idealised footprint were confirmed by t-bar characterisation tests.

3.4.2 Penetration Velocity

The penetration rate of penetrometers and jack-up leg was of a rate fast enough to ensure that undrained condition was achieved. Using the framework of Finnie (1993), the normalised penetration velocity can be deduced as (3.3)

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

(3.3)

where \( v \) is the penetration velocity, \( D \) the diameter and \( c_v \) the coefficient of consolidation. \( V \) is the normalised penetration velocity and should not be less than 30 to ensure undrained condition. However, high penetration rates cause viscous effect. Therefore, based on Low et al. (2008) a maximum normalised penetration rate of 120 was proposed as the upper bound. The penetration velocity adopted for the t-bar and ball penetrometer was 1mm/s and for the footings was 0.1mm/s. This produced \( V=78.84 \) and \( V=94.61 \) respectively, and both fulfilled the criteria.

3.4.3 Testing Programme

Two t-bar characterisation tests were conducted in the undisturbed soil and the location of test sites are shown in Figure 3.14. One footing installation test through the flat surface and 16 reinstallation tests next to idealised footprints were conducted on the drum centrifuge sample. These are summarised in Table 3.7, with the testing layout shown in Figure 3.14. In order to minimise the boundary effect, the minimum distance between test sites was 1.5 times the footing diameter.
The nomenclature of the tests is in the form of “footprint type-footprint diameter to footing diameter ratio-offset distance as a ratio to D-remarks” (“footprint type-x_F/D-β-remarks”). The decimal place is omitted. For example, for a footing reinstallation test with flat-base footing installed at 0.5D from footprint TA of 180mm diameter, the name of the test is therefore TA-3D-05D. The remarks term was only applicable to skirted footing tests and for repeated tests. The layout of the tests in the drum centrifuge channel is shown in Figure 3.15.

Table 3.7: List of Footing Installation and Reinstallation Tests

<table>
<thead>
<tr>
<th>Footprint Type</th>
<th>Normalised Dimension</th>
<th>β</th>
<th>Test Name</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>x_F/D</td>
<td>z_F/D</td>
<td></td>
</tr>
<tr>
<td>TA</td>
<td>3</td>
<td>0.25</td>
<td>1.0D</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.17</td>
<td>1.0D</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.5D</td>
</tr>
<tr>
<td>TB</td>
<td>2</td>
<td>0.33</td>
<td>0.5D</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.0D</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.5D</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>0.5</td>
<td>0.5D</td>
</tr>
<tr>
<td></td>
<td>1.33</td>
<td>0.75</td>
<td>0.75D</td>
</tr>
<tr>
<td>TC</td>
<td>2</td>
<td>0.67</td>
<td>0.5D</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.0D</td>
</tr>
<tr>
<td>SB</td>
<td>2</td>
<td>0.33</td>
<td>1.0D</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.5D</td>
</tr>
<tr>
<td>TB – Skirted</td>
<td>2</td>
<td>0.33</td>
<td>1.0D</td>
</tr>
<tr>
<td>Footing</td>
<td></td>
<td></td>
<td>1.5D</td>
</tr>
<tr>
<td>TB – Control</td>
<td>2</td>
<td>0.33</td>
<td>1.0D</td>
</tr>
<tr>
<td>Test</td>
<td></td>
<td></td>
<td>1.0D</td>
</tr>
<tr>
<td>Flat Surface</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Chapter 3: Geometric Effect of Footprint

Figure 3.14: Layout Plan of T-bar/Ball Characterisation Tests and Footing Reinstallation Tests
3.5 EXPERIMENTAL RESULTS

In this section, the laser profiling result will firstly be presented to verify the footprint geometry. The t-bar characterisation test results are also interpreted to confirm the undrained shear strength profile of the soil sample. The experimental results from the footing installation test (FS) and an example of reinstallation test (TB-2D-10D) will also be presented in this section before a detailed comparison and discussion in Section 3.6.

3.5.1 Footprint Profile

Shown in Figure 3.16 are the vertical and radial laser profiles of the footprint left after test FS. The footprint was created with a maximum vertical force of 60MN, corresponding to a maximum preload pressure of 339kPa. The ground profile for this real footprint had a fairly gentle sloping ground surface, with $x_F$ of 2D and $z_F$ of 0.2D. As shown in Figure 3.16, the footprint profiles are very similar to those presented in Cassidy et al. (2009) and Gan (2008a).

The recorded footprint profile lies between the idealised footprint TA and TB profiles, where as footprint TC is deeper than all the recorded footprint profile and can be considered as an upper bound. However, all three idealised footprint cavity types are confirmed as good representations of real footprint profiles and bound the possible footprint geometries.
3.5.2 Undrained shear strength of Soil Sample

The undrained shear strength profile is related to the stress history and can be estimated using the empirical relationship of Ladd et al. (1977)

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

(3.4)

where OCR is the over-consolidation ratio and \( \sigma'_v \) the effective vertical stress. For UWA kaolin clay in the centrifuge, the equation parameter \( a \) is in the range 0.17 to 0.185 and \( n \) 0.7 to 0.8 (Stewart, 1992, Teh et al. 2007). Therefore, \( a=0.17 \) and \( n=0.7 \) were adopted in this study.

3.5.2.1 Undrained shear strength of Undisturbed Soil Sample

The undrained shear strength profile of the over-consolidated kaolin clay soil sample was determined from the two t-bar tests and the results are presented in Figure 3.17. The results exhibited good consistency and can be described by two linear relationships:

\[ s_u = s_{um} + k z \]  

(3.5)

where \( s_u \) is the undrained shear strength, \( s_{um} \) is the undrained shear strength at soil surface, \( k \) is the rate of increase in undrained shear strength with depth and \( z \) is the depth below the soil surface. The following bi-linear relationships are proposed to estimate the soil strength in prototype dimension:
\[ s_u = 7.5 + 0.92z, \text{ for } z < 3.4m \]  
(3.6)

\[ s_u = 5.0 + 1.68z, \text{ for } z \geq 3.4m \]  
(3.7)

3.5.2.1 Shear strength of Idealised Footprint

The undrained shear strength, \( s_u \) under the idealised footprint cavity was also investigated using the ball penetrometer. A type TC footprint has been created to determine the undrained shear strength profile underneath an idealised footprint cavity. Characterisation tests were conducted 24 hours (model dimension) after the creation of the footprint, which was the same elapsed time adopted in the footing reinstallation test. Figure 3.18 compares the theoretical undrained shear strength with the measured \( s_u \) values. The largest difference was at the footprint crest. However, the agreement improves with depth and with distance from the footprint toe. The measured values are in general lower than the theoretical values and the maximum difference is approximately 3kPa. The difference is relatively small and the trend of change in measured undrained shear strength with depth and distance is in good agreement with the theoretical profile. Therefore, the methodology adopted in creating idealised footprint cavity was considered to be successful in minimising soil disturbance beneath the footprint.


3.5.3 Footing Installation Test

The results of footing installation test FS are presented in Figure 3.19. The vertical load increased drastically from touchdown level as the footing came into contact with the founding soil. The gradient then reduced, with the vertical load increasing linearly until reaching the maximum penetration depth. Slight local variations at approximately 6m and 16m were observed and this is likely due to the soft layer formed by the sequence of placement of kaolin slurry during the clay sample preparation.

The bearing capacity factor \( N_c \) back-calculated from the experimental results are also shown in Figure 3.19. \( N_c \) is defined in equation A.4. It is noted that the \( N_c \) gradually increased and reached a constant value of 9.2 at normalised depth \((z/D)\) below 0.55. This confirmed that the constant increase in penetration resistance is related to the linear increase in undrained shear strength with depth.
3.5.4 Typical Footing Reinstallation Test

The results for one typical footing reinstallation test are presented in Figure 3.20. Also shown, for comparison, are the results for the footing installation in flat surface without footprint, test FS. The horizontal and moment load at the footing were evaluated from the strain gauges, as discussed in Section 3.3.4 and with reference to the sign convention presented in Figure 3.9. The test shown is TB-2D-10D, which was a flat-base footing installed at 1.0D offset distance from the centre of the footprint cavity TB ($x_F/D=2$ and $z_F/D=0.33$). Positive horizontal force indicates that the footing was resisting a movement towards the footprint and a positive moment indicates the footing was resisting a counter-clockwise rotation away from the footprint. The directions of soil loading agreed with the results presented in other publications (Stewart and Finnie 2001, Cassidy et al. 2009, Gan 2009).

The measured response can be considered to be in three stages: 1) at touchdown ($z/D\leq0.05$), 2) below touchdown to the toe of footprint ($0.05<z/D\leq0.33$) and 3) below the toe of the footprint ($z/D>0.33$). In stage 1, TB-2D-10D exhibited a lower vertical force compared to FS. While there was no horizontal force acting on the footing, the moment increased rapidly and reached its maximum value ($M_{\text{max}}$) almost immediately at touchdown level. The soil was trying to rotate the footing away from the idealised footprint cavity. In stage 2, the vertical force was still lower than the FS test due to the influence of the idealised footprint cavity. Below the footprint toe level, the vertical force eventually became similar to penetration in a flat surface. The horizontal force increased gradually with penetration until reaching a peak value ($H_{\text{max}}$) at $z/D=0.33$, indicating that the soil was pushing the footing towards the idealised footprint cavity. Moment reduced to almost zero at the end of stage 2. In stage 3, both the horizontal

![Figure 3.19: Vertical Responses and Bearing Capacity Factor of Footing Installation in Flat Surface](image-url)
force and moment fluctuated slightly before reaching zero. The simplified three-stage responses are shown in Figure 3.21.
Figure 3.20: The Response of Footing Installed at 1.0D Offset from Footprint TB (Vertical, Horizontal and Moment Responses)
To investigate the combined loading acting on the footing, an inclination angle ($\alpha$) and dimensionless eccentricity ($e/D$) are defined as:

$$\alpha = \tan^{-1}\left(\frac{H}{V}\right)$$  \hfill (3.8)

$$\frac{e}{D} = \frac{M}{VD}$$  \hfill (3.9)

These have been shown in Figure 3.20 for both the TB-2D-10D and FS tests. The peak responses are now more clearly defined and occurred within stage 1 (peak eccentricity) and stage 2 (peak inclination). The relatively small variation in $\alpha$ and $e/D$ confirmed that the non-zero moment and horizontal force below footprint toe are insignificant. Therefore, the response from touchdown to the toe of footprint (stages 1 and 2) are considered to be critical in understanding the influence of footprint geometry on footing reinstallation.

The vertical force was normalised by the footing area, $A$ and the undrained shear strength at the position of the bottom of the footing, $s_u$ to assess the reduction in vertical capacity during reinstallation. $N_c$ which is defined for bearing capacity under purely vertical load is only applicable to test FS. Therefore, for a reinstallation test for which
the footing is under combined vertical-horizontal-moment loadings, the vertical force normalised by $A_{su}$ does not equal $N_c$. Figure 3.22 shows two different normalised vertical force graphs for TB-2D-10D together with the $N_c$ of the FS. It is difficult to determine the change in contact area during the test as the idealised footprint cavity deforms with penetration. Only half of the footing was in contact with soil and the contact area gradually increases with penetration depth and deformation of the idealised footprint cavity until the footing came into full contact with the soil. Therefore, the vertical force was normalised by the full area of the footing (labelled “Full Area”) and by the contact area assuming that the idealised footprint cavity remained intact throughout penetration (labelled “Contact Area”). Through this, the lower bound and upper bound of normalised vertical force could be evaluated.

In stage 1, the normalised vertical force of TB-2D-10D is 50% of FS. The normalised vertical force increased rapidly until at $z/D$ of 0.17. Although the footing had already come into full contact with the soil at this depth, the normalised vertical force remained lower and increased gradually to finally match the FS at depth below the toe of footprint. The lower vertical capacity during footprint reinstallation is related to the combined loading condition. The lower soil strength of the deformed footprint would also contribute to the reduction in vertical capacity.

![Figure 3.22: The Normalised Vertical Force of Footing Installed at 1.0D Offset from Footprint TB](image-url)
The simplified three-stage reinstallation response shown in Figure 3.21 were compared with the results shown in Gan (2009) concerning reinstallation near real footprints. The reinstallation responses near footprints in normally-consolidated and over-consolidated soils are shown in Figure 3.23 and Figure 3.24.

The three-stage reinstallation response is similar to the reinstallation response near a real footprint with long elapsed time (ET>100 years). A distinct peak in the horizontal force and moment was observed within the touchdown and \( z_F \). The horizontal force and moment reduced to zero until further penetration. The relatively gentle increase in the moment near the touchdown level was likely to be related to the lower soil strength in a real footprint.

For reinstallation with relatively short elapsed time (ET=1 or 3 years), the horizontal force and moment increased from touchdown level and reached the maximum values at 0.6-0.8D instead of near \( z_F \). It is because the soil could not regain its full strength within the relatively short elapsed time and the soil strength around footprint was highly variable. Therefore the reinstallation response was controlled by the combined effect of footprint geometry and soil heterogeneity.

During reinstallation near a real footprint, a negative peak horizontal force and moment was observed near \( z_{FC} \) (maximum penetration depth during footprint creation) and it was related to the strong soil layer formed near \( z_{FC} \). This deep reinstallation response was controlled by the soil heterogeneity rather than by the footprint geometry. Further discussion on the deep reinstallation response is presented in Section 8.4.2.

In summary, considering the footprint geometry in isolation provided a good approximation to the shallow depth responses during footing reinstallation in either normally-consolidated or over-consolidated clay with long elapsed time.
Figure 3.23: Horizontal Force and Moment During Reinstallation Near a Real Footprint in Normally-Consolidated Clay (after Gan 2009)
3.5.5 Summary of Experimental Results

The peak loadings \((H_{\text{max}}, M_{\text{max}}, a_{\text{max}} \text{ and } c/D_{\text{max}})\) are summarised in Table 3.8. In addition, the depths where maximum horizontal force and moment occurred were normalised by the footing diameter \((z_{H_{\text{max}}}/D \text{ and } z_{M_{\text{max}}}/D)\) and presented in Table 3.8. Also presented are the maximum values of normalised horizontal force and moment (normalised by the full area of footing and the undrained shear strength of soil at the depth of penetration). This allows direct comparison with other published data. Detailed discussions and comparisons of the test results are presented in Section 3.6.
### Table 3.8: Summary of Peak Responses from Footing Reinstallation Test near Idealised Footprint Cavity (Refer to Table 3.7 for the Naming System of Tests)

<table>
<thead>
<tr>
<th>Test</th>
<th>$H_{\text{max}}$ (MN)</th>
<th>$z_{H_{\text{max}}}/D$ (m)</th>
<th>$(H/As_u)_{\text{max}}$</th>
<th>$a_{\text{max}}$ (°)</th>
<th>$M_{\text{max}}$ (MNm)</th>
<th>$z_{M_{\text{max}}}/D$ (m)</th>
<th>$(M/As_uD)_{\text{max}}$</th>
<th>$(e/D)_{\text{max}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>TA-3D-10D</td>
<td>0.76</td>
<td>0.18</td>
<td>0.43</td>
<td>3.48</td>
<td>16.43</td>
<td>0.05</td>
<td>0.74</td>
<td>0.27</td>
</tr>
<tr>
<td>TA-2D-10D</td>
<td>0.46</td>
<td>0.17</td>
<td>0.26</td>
<td>1.52</td>
<td>10.46</td>
<td>0.03</td>
<td>0.49</td>
<td>0.11</td>
</tr>
<tr>
<td>TA-2D-15D</td>
<td>0.32</td>
<td>0.16</td>
<td>0.19</td>
<td>1.25</td>
<td>3.89</td>
<td>0.03</td>
<td>0.18</td>
<td>0.02</td>
</tr>
<tr>
<td>TB-2D-05D</td>
<td>0.73</td>
<td>0.25</td>
<td>0.38</td>
<td>5.46</td>
<td>15.79</td>
<td>0.17</td>
<td>0.63</td>
<td>0.36</td>
</tr>
<tr>
<td>TB-2D-10D</td>
<td>0.88</td>
<td>0.31</td>
<td>0.40</td>
<td>2.84</td>
<td>11.91</td>
<td>0.05</td>
<td>0.54</td>
<td>0.19</td>
</tr>
<tr>
<td>TB-2D-10D-R</td>
<td>0.94</td>
<td>0.27</td>
<td>0.47</td>
<td>3.78</td>
<td>13.91</td>
<td>0.01</td>
<td>0.62</td>
<td>0.23</td>
</tr>
<tr>
<td>TB-2D-10D-R2</td>
<td>0.98</td>
<td>0.28</td>
<td>0.45</td>
<td>3.07</td>
<td>17.03</td>
<td>0.09</td>
<td>0.70</td>
<td>0.29</td>
</tr>
<tr>
<td>TB-2D-15D</td>
<td>0.46</td>
<td>0.21</td>
<td>0.26</td>
<td>2.00</td>
<td>3.80</td>
<td>0.01</td>
<td>0.16</td>
<td>0.02</td>
</tr>
<tr>
<td>TB-1D-05D</td>
<td>0.31</td>
<td>0.08</td>
<td>0.20</td>
<td>1.57</td>
<td>11.13</td>
<td>0.03</td>
<td>0.52</td>
<td>0.13</td>
</tr>
<tr>
<td>TB-13D-075D</td>
<td>0.42</td>
<td>0.17</td>
<td>0.24</td>
<td>1.76</td>
<td>13.24</td>
<td>0.03</td>
<td>0.62</td>
<td>0.17</td>
</tr>
<tr>
<td>TB-2D-10D-S</td>
<td>2.16</td>
<td>0.26</td>
<td>1.13</td>
<td>4.08</td>
<td>11.47</td>
<td>0.04</td>
<td>0.53</td>
<td>0.10</td>
</tr>
<tr>
<td>TB-2D-15D-S</td>
<td>1.99</td>
<td>0.28</td>
<td>0.95</td>
<td>2.85</td>
<td>3.68</td>
<td>0.01</td>
<td>0.11</td>
<td>0.01</td>
</tr>
<tr>
<td>TC-2D-05D</td>
<td>1.73</td>
<td>0.40</td>
<td>0.67</td>
<td>19.60</td>
<td>25.17</td>
<td>0.40</td>
<td>0.64</td>
<td>0.50</td>
</tr>
<tr>
<td>TC-2D-10D</td>
<td>1.21</td>
<td>0.20</td>
<td>0.69</td>
<td>22.93</td>
<td>16.26</td>
<td>0.20</td>
<td>0.61</td>
<td>0.42</td>
</tr>
<tr>
<td>SB-2D-10D</td>
<td>0.96</td>
<td>0.22</td>
<td>0.51</td>
<td>3.64</td>
<td>12.72</td>
<td>0.03</td>
<td>0.57</td>
<td>0.27</td>
</tr>
<tr>
<td>SB-2D-15D</td>
<td>0.51</td>
<td>0.27</td>
<td>0.22</td>
<td>1.13</td>
<td>6.60</td>
<td>0.26</td>
<td>0.24</td>
<td>0.02</td>
</tr>
</tbody>
</table>

### 3.6 COMPARISON AND DISCUSSION ON EXPERIMENTAL RESULTS

In this section, quantitative comparisons that assess the critical factors in the response of a footing reinstallation are presented. Discussions on the failure mechanism are also made based on the loading-penetration depth relation.
3.6.1 Control Tests Results

Among the 16 footing reinstallation tests, 2 control tests were conducted and the results are summarised in Table 3.9.

TB-2D-10D-R was a repeated test carried out 7 days after the completion of TB-2D-10D. The testing conditions of the two tests were identical. The second control test TB-2D-10D-R2 was conducted at a reduced acceleration level of 187.5g. This is a model scaling test, as it reduces the gravity level to the same degree it increases the footing size, with the prototype footing equivalent to TB-2D-10D. The idealised footprint cavity geometry was also increased in magnitude so that all prototype dimensions were identical to that of TB-2D-10D. TB-2D-10D-R2 was conducted 8 days after the completion of TB-2D-10D. As shown in Figure 3.25, the measured responses are consistent and the difference in \((H/As_u)_{\text{max}}\) and \((M/As_u)_{\text{max}}\) is around 10% and 20% respectively. The slight increase in the soil strength due to the reconsolidation process over the 7-8 days period between the tests is considered to be the main reason for the difference. The magnitude of variation was considered to be acceptable, for this type of centrifuge testing, and the repeatability provided confidence in the testing procedures.

Table 3.9: Summary of Maximum Loads Recorded During Control Test

<table>
<thead>
<tr>
<th>Test</th>
<th>Maximum Horizontal Load</th>
<th>Maximum Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(H_{\text{max}}) (MN)</td>
<td>(H/As_u)_{\text{max}})</td>
</tr>
<tr>
<td>TB-2D-10D</td>
<td>0.88</td>
<td>0.40</td>
</tr>
<tr>
<td>TB-2D-10D-R</td>
<td>0.94</td>
<td>0.47</td>
</tr>
<tr>
<td>TB-2D-10D-R2</td>
<td>0.98</td>
<td>0.45</td>
</tr>
</tbody>
</table>
Figure 3.25: The Response of Footings Reinstalled Near Footprint TB – Control Tests
3.6.2 Effect of Footprint Size (Combined Effect of Footprint Depth and Slope Angle)

The results are presented by footprint size in Table 3.10 and Figure 3.26 to Figure 3.28. Footings installed next to the deepest and steepest footprint TC produced the largest results. Consistently over all the offset distances, the $H_{\text{max}}$ in reinstallation near footprint TC is three times higher. However, the increase in $M_{\text{max}}$ is more subtle. When $\beta=1.0$ is considered, the $M_{\text{max}}$ increased by 20% when the footprint type changed from TA to TC. Figure 3.29 shows that there is also an increasing trend in normalised peak loadings with increasing footprint size. The footprint size, in terms of footprint depth and footprint angle, is confirmed as critical to the peak loadings induced in a footing reinstallation.

Table 3.10: Maximum Loads from Footing Reinstallation Test near Footprints of Different Size

<table>
<thead>
<tr>
<th>Test</th>
<th>$\beta$</th>
<th>$H_{\text{max}}$ (MN)</th>
<th>$(H/As_u)_{\text{max}}$</th>
<th>$\alpha_{\text{max}}$ (°)</th>
<th>$M_{\text{max}}$ (MNm)</th>
<th>$(M/AsuD)_{\text{max}}$</th>
<th>$(e/D)_{\text{max}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>TB-2D-05D</td>
<td>0.5D</td>
<td>0.73</td>
<td>0.38</td>
<td>5.46</td>
<td>15.79</td>
<td>0.63</td>
<td>0.36</td>
</tr>
<tr>
<td>TC-2D-05D</td>
<td></td>
<td>1.73</td>
<td>0.67</td>
<td>19.60</td>
<td>25.17</td>
<td>0.64</td>
<td>0.50</td>
</tr>
<tr>
<td>TA-2D-10D</td>
<td>1.0D</td>
<td>0.46</td>
<td>0.26</td>
<td>1.52</td>
<td>10.46</td>
<td>0.49</td>
<td>0.11</td>
</tr>
<tr>
<td>TB-2D-10D</td>
<td></td>
<td>0.88</td>
<td>0.40</td>
<td>2.84</td>
<td>11.91</td>
<td>0.54</td>
<td>0.19</td>
</tr>
<tr>
<td>TC-2D-10D</td>
<td></td>
<td>1.21</td>
<td>0.69</td>
<td>22.93</td>
<td>16.26</td>
<td>0.61</td>
<td>0.42</td>
</tr>
<tr>
<td>TA-2D-15D</td>
<td>1.5D</td>
<td>0.32</td>
<td>0.19</td>
<td>1.25</td>
<td>3.89</td>
<td>0.18</td>
<td>0.02</td>
</tr>
<tr>
<td>TB-2D-15D</td>
<td></td>
<td>0.46</td>
<td>0.26</td>
<td>2.00</td>
<td>3.80</td>
<td>0.16</td>
<td>0.02</td>
</tr>
</tbody>
</table>
Figure 3.26: The Response of Footings Reinstalled Near Footprint TA
Figure 3.27: The Response of Footings Reinstalled Near Footprint TB
Chapter 3: Geometric Effect of Footprint

Figure 3.28: The Response of Footings Reinstalled Near Footprint TC
Chapter 3: Geometric Effect of Footprint

Figure 3.29: Comparison of Maximum Loads from a Reinstallation Test Next to Footprints of Different Size

Figure 3.30 and Figure 3.31 show that there is also a consistent increase in both $\alpha_{\text{max}}$ and $(e/D)_{\text{max}}$ with change in footprint size. A similar non-linear trend of increasing $\alpha_{\text{max}}$ with footprint size was observed over the three offset distances considered. The non-linearity increased with reducing offset distance. Figure 3.31 shows that $(e/D)_{\text{max}}$ increased linearly when footprint type changed from TA to TC. The rate of increase in $(e/D)_{\text{max}}$ increased with reducing offset distance.

Figure 3.30: Comparison of Maximum Inclination Angle from a Footing Reinstallation test Next to Footprints of Different Size
3.6.3 Effect of Offset Distance

To identify the critical offset distance, $\beta$, the results of footing reinstallation tests at $\beta$ of 0.5D, 1.0D and 1.5D are compared in Figure 3.32. The $H_{\text{max}}$ and $M_{\text{max}}$ generally reduce with increasing $\beta$. The $H_{\text{max}}$ values for footprint TB, however, shows a peak at $\beta=1.0D$.

The maximum normalised horizontal force, $(H/As_u)_{\text{max}}$ and maximum inclination angle, $\alpha_{\text{max}}$ are shown in Table 3.11 and Table 3.12. The maximum normalised moment, $(M/As_uD)_{\text{max}}$ and the maximum eccentricity, $(e/D)_{\text{max}}$ are shown in Table 3.13 and Table 3.14. In normalised values the $\beta$ of 1.0D is the most critical. The loads recorded for 1.5D are minimal and this may provide a bound for when the effect of the footprint geometry tapers off.

Also shown in Table 3.11 to Table 3.14 are experimental results from four previous studies: Teh et al. (2006), Gaudin et al. (2007), Cassidy et al. (2009) and Gan (2009). When increasing $\beta$ from 0.5D to 1.5D, the reduction in $(H/As_u)_{\text{max}}$ is only around 30% instead of the factor of 2 to 3 reported in all the publications. However, the magnitude of the $(H/As_u)_{\text{max}}$ response for footprint TA and TB are comparable with the other publications, especially with Stewart and Finnie (2001) and Gan (2009). The footprint geometry, therefore, dominated the $H_{\text{max}}$ response.

The $(M/As_uD)_{\text{max}}$ reduced by a factor of 3 when $\beta$ increased from 1.0D to 1.5D which is consistent with the finding of Stewart and Finnie (2001) and Cassidy et al. (2009). The role of offset distance is shown to be more critical to the bending resistance than the horizontal resistance. However, the $(M/As_uD)_{\text{max}}$ response for footprint TA and TB are
generally higher than the other test data. A possible reason is that higher bearing resistance was mobilised from the idealised footprint cavity than the real footprint with disturbed soil. Therefore, moment increased with this eccentric bearing resistance. Teh et al. (2006) was based on 1g-scaled model test and provided a lower bound.

Figure 3.32: Comparison of Maximum Loads from a Footing Reinstallation Test at Different Offset Distance

Table 3.11: Comparison of Maximum Normalised Horizontal Force from Footing Reinstallation Test near Different Footing Types with Other Studies

<table>
<thead>
<tr>
<th>β</th>
<th>(H/Asu) max</th>
<th>(H/Asu) max</th>
<th>(H/Asu) max</th>
<th>(H/Asu) max</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>TA</td>
<td>TB</td>
<td>TC</td>
<td>TA</td>
</tr>
<tr>
<td>0.5D</td>
<td>0.29</td>
<td>0.57</td>
<td>0.07</td>
<td>0.38</td>
</tr>
<tr>
<td>1.0D</td>
<td>0.38</td>
<td>0.70</td>
<td>-</td>
<td>0.44</td>
</tr>
<tr>
<td>1.5D</td>
<td>0.18</td>
<td>0.20</td>
<td>-</td>
<td>0.16</td>
</tr>
</tbody>
</table>
Table 3.12: Comparison of Maximum Inclination Angle from Footing Reinstallation Test near Different Footing Type with Other Studies

<table>
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<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>TA</td>
<td>TB</td>
<td>TC</td>
<td></td>
</tr>
<tr>
<td>0.5D</td>
<td>2.32</td>
<td>8.20</td>
<td>2.35</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>5.46</td>
<td>19.60</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.0D</td>
<td>2.69</td>
<td>5.30</td>
<td>-</td>
<td>1.52</td>
</tr>
<tr>
<td></td>
<td>2.84</td>
<td>22.93</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5D</td>
<td>1.15</td>
<td>1.00</td>
<td>-</td>
<td>1.25</td>
</tr>
<tr>
<td></td>
<td>2.00</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3.13: Comparison of Maximum Normalised Moment from Footing Reinstallation Test near Different Footing Type with Other Studies

<table>
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</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>TA</td>
<td>TB</td>
<td>TC</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5D</td>
<td>0.19</td>
<td>0.54</td>
<td>0.22</td>
<td>0.11</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>0.63</td>
<td>0.64</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.0D</td>
<td>0.22</td>
<td>0.67</td>
<td>-</td>
<td>0.21</td>
<td>0.49</td>
</tr>
<tr>
<td></td>
<td>0.54</td>
<td>0.61</td>
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<td></td>
<td></td>
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<tr>
<td>1.5D</td>
<td>0.04</td>
<td>0.14</td>
<td>-</td>
<td>-</td>
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</tr>
<tr>
<td></td>
<td>0.16</td>
<td></td>
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</table>

Table 3.14: Comparison of Maximum Eccentricity from Footing Reinstallation Test near Different Footing Type with Other Studies

<table>
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<tr>
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</thead>
<tbody>
<tr>
<td></td>
<td>TA</td>
<td>TB</td>
<td>TC</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5D</td>
<td>0.047</td>
<td>0.17</td>
<td>0.13</td>
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<td>-</td>
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<tr>
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<tr>
<td>1.0D</td>
<td>0.047</td>
<td>0.09</td>
<td>-</td>
<td>3.54</td>
<td>0.11</td>
</tr>
<tr>
<td></td>
<td>0.19</td>
<td>0.42</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5D</td>
<td>0.007</td>
<td>0.02</td>
<td>-</td>
<td>-</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>0.02</td>
<td></td>
<td></td>
<td></td>
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</table>

Further comparisons are presented in Figure 3.33 and Figure 3.34, the critical β obtained from this study is in good agreement with these publications, which suggests that the critical reinstallation distance is governed by the footprint geometry, rather than by the remoulded soil strength of the footprint. In summary, β=1.0D, which involved half of the footing in contact with the soil, appeared to be the most critical offset distance.
3.6.4 Effect of Footprint Diameter

As presented in Section 2.2.3, Gan (2009) found that the peak reinstallation loads increased with diameter ratio, which is the diameter of the spudcan creating the footprint (D_f) to diameter of the reinstalling spudcan (D). Therefore, the effect of diameter ratio (D_f/D) is investigated here to confirm the controlling parameters being the footprint geometry or footprint soil strength. The investigation considered two TA footprints of different diameter, as shown in Figure 3.35. Test TA-3D-10D adopted a 45m diameter footprint. Based on the assumption that the footprint diameter (D_f) equals two times the diameter of the footing creating the footprint, the test therefore considered a footprint created by a 22.5m diameter spudcan (90mm in model dimension). The
reinstalling footing was 15m in diameter, therefore the ratio, $D_f/D$ was 1.5. The footprint depth ($z_F$) also increased proportionally so that the footprint slope angle was the same and direct comparison can be made between test results. The same offset distance ($\beta=15\text{m}$) was adopted to investigate the isolated effect of footprint diameter. The test results are shown in Figure 3.36.

![Figure 3.35: Footprints in Different Diameter](image)

Table 3.15 shows that the normalised peak loadings of tests conducted in the larger footprint are 2 times larger than in standard TA footprint. This is consistent with Gan (2009) which reported a 50% increase the $(H/As_u)_{\text{max}}$ and 35% increase in the $(M/As_uD)_{\text{max}}$ when $D_f/D$ increased from 1.0 to 1.33. Therefore, the increase in peak loadings with $D_f/D$ was due to the geometry. As $D_f/D$ increased, the footing reinstalled nearby the larger footprint experienced larger loads. The diameter of footprints (therefore $D_f/D$) is considered to be a critical parameter to the responses of footing reinstallation. During reinstallation it is desirable to adopt a jack-up unit with a footing diameter larger than or equal to the one adopted in the previous installation.
Figure 3.36: The Response of Footing Reinstalled Near Footprint TA
Table 3.15: Comparison of Maximum Loads from Footing Reinstallation Tests near Footprint TA with Different Diameter Ratios

<table>
<thead>
<tr>
<th>Test</th>
<th>Df/D</th>
<th>xF/D</th>
<th>Hmax (MN)</th>
<th>(H/Asu)max</th>
<th>a_max (°)</th>
<th>Mmax (MNm)</th>
<th>(M/AsuD)max</th>
<th>(e/D)max</th>
</tr>
</thead>
<tbody>
<tr>
<td>TA-3D-10D</td>
<td>1.5</td>
<td>3</td>
<td>0.76</td>
<td>0.43</td>
<td>3.48</td>
<td>16.43</td>
<td>0.74</td>
<td>0.27</td>
</tr>
<tr>
<td>TA-2D-10D</td>
<td>1.0</td>
<td>2</td>
<td>0.46</td>
<td>0.26</td>
<td>1.52</td>
<td>10.46</td>
<td>0.49</td>
<td>0.11</td>
</tr>
</tbody>
</table>

3.6.5 Effect of Footprint Depth

Section 3.6.3 shows that reinstallation with half of the footing resting on the founding soil gave the most critical peak loadings. This critical situation was further explored in combination with the effect of footprint depth. Modification was made to footprint TB to consider footprints with three depths (zF/D=0.17, 0.22 and 0.33) as shown in Figure 3.37. The offset distance β was adjusted so only half of the footing was resting on the founding soil (the critical situation). As a result the contact area at the touchdown level was slightly different (within 15% difference). The geometric parameters of footprints are shown in Table 3.16.

![Figure 3.37: Footing Reinstallation at Critical Offset Distance from Footprints with Different Diameter Ratios](image-url)
Table 3.16: Parameters of Footprints with Different Diameter Ratios

<table>
<thead>
<tr>
<th>Test</th>
<th>Contact Area at Touchdown (mm²)</th>
<th>D/D</th>
<th>Footprint Parameters (Model Dimension)</th>
<th>Normalised Dimension</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>x_F (mm)</td>
<td>z_F (mm)</td>
</tr>
<tr>
<td>TB-1D-05D</td>
<td>1700</td>
<td>0.5</td>
<td>60</td>
<td>10mm</td>
</tr>
<tr>
<td>TB-13D-075D</td>
<td>1900</td>
<td>0.67</td>
<td>80</td>
<td>13mm</td>
</tr>
<tr>
<td>TB-2D-10D</td>
<td>1600</td>
<td>1.0</td>
<td>120</td>
<td>20mm</td>
</tr>
</tbody>
</table>

The experimental results are presented in Figure 3.38 and the peak responses are compared in Figure 3.39. When z_F/D was increased from 0.17 to 0.33, the H_max and H_max/As_u increased linearly. The H_max/As_u during reinstallation near the deepest footprint (z_F/D=0.33) was two times larger than the shallowest one (z_F/D=0.17). Although the moment also exhibited an increasing trend with footprint depth, the difference in M_max/As_uD between the three tests was within 20% only. The effect of footprint depth was therefore more influential to the H_max but not to the M_max.
Figure 3.38: The Response of Footing Reinstalled Near Modified Footprint TB
3.6.6 Sloped Surface

Comparison was made to the test results on footprint TB and slope SB to investigate the three-dimensional effect of an idealised footprint cavity. The angle of the footprint and slope are identical. Two offset distances of $\beta=1.0D$ and $1.5D$ were investigated, with results provided in Figure 3.40 and Figure 3.41.
Figure 3.40: The Response of Footings Reinstalled at 1.0D Offset from Footing TB and from Sloping Ground
Figure 3.41: The Response of Footings Reinstalled at 1.5D Offset from Footing TB and from Sloping Ground
Table 3.17 summarises the peak responses. The difference in the peak loading for $\beta=1.0D$ is around 20%. The variation is likely to be coming from the change in soil strength profiles, as the tests were carried out on different days. The overall trend of the measured load response is found to be similar. Therefore, the three-dimensional effect of a slope compared to a footprint is considered to be relatively insignificant.

Table 3.17: Comparison of Maximum Loads from Footing Reinstallation Tests near Footprint TB and a Near Slope

<table>
<thead>
<tr>
<th>Test</th>
<th>Footprint Type/ Slope</th>
<th>$\beta$</th>
<th>Maximum Horizontal Load</th>
<th>Maximum Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$H_{\text{max}}$ (MN)</td>
<td>$M_{\text{max}}$ (MNm)</td>
</tr>
<tr>
<td>TB-2D-10D</td>
<td>TB-2D</td>
<td>1.0D</td>
<td>0.88</td>
<td>11.91</td>
</tr>
<tr>
<td>SB-2D-10D</td>
<td>Slope</td>
<td></td>
<td>0.96</td>
<td>12.72</td>
</tr>
<tr>
<td>TB-2D-15D</td>
<td>TB-2D</td>
<td>1.5D</td>
<td>0.46</td>
<td>3.80</td>
</tr>
<tr>
<td>SB-2D-15D</td>
<td>Slope</td>
<td></td>
<td>0.51</td>
<td>6.60</td>
</tr>
</tbody>
</table>

3.6.7 Skirted Footing

Skirted footings have been considered by industry as a potential mitigation measures for jack-up reinstallations near a footprint (Teh et al. 2006). Two tests were conducted on this soil sample to investigate their reinstallation responses and the results are compared with flat-base footing tests (Figure 3.42 and Figure 3.43). Similar to the case of flat-base footing, $M_{\text{max}}$ and $(e/D)_{\text{max}}$ occurred near the touchdown level and $H_{\text{max}}$ and $\alpha_{\text{max}}$ occurred when the penetration depth was near the footprint toe. Surprisingly, there was no significant different between the $M_{\text{max}}$ resulted from reinstallation of the two different footing types. On-the-other-hand, the $H_{\text{max}}$ increased by a factor of two compared to a flat-base footing when the skirted footing was adopted (Table 3.18). The normalised peak loading, $(H/As_u)_{\text{max}}$ and $(M/As_uD)_{\text{max}}$ of skirted footing reduced in a similar ratio as for flat-base footing when the $\beta$ increased from 1.0D to 1.5D.
Figure 3.42: The Response of Footings Reinstalled at 1.0D Offset from Footing TB (Flat-Base Footing and Skirted Footing)
Figure 3.43: The Response of Footings Reinstalled at 1.5D Offset from Footing TB (Flat-Base Footing and Skirted Footing)
Table 3.18: Comparison of Maximum Loads from Footing Reinstallation Tests near Footprint TB and near a Slope

<table>
<thead>
<tr>
<th>Test</th>
<th>Footing Type</th>
<th>β</th>
<th>$H_{\text{max}}$ (MN)</th>
<th>$(H/\text{As}<em>u)</em>{\text{max}}$</th>
<th>$\alpha_{\text{max}}$ (°)</th>
<th>$M_{\text{max}}$ (MNm)</th>
<th>$(M/\text{As}<em>uD)</em>{\text{max}}$</th>
<th>$(e/D)_{\text{max}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>TB-2D-10D</td>
<td>Flat-base footing</td>
<td>1.0D</td>
<td>0.88</td>
<td>0.40</td>
<td>2.84</td>
<td>11.91</td>
<td>0.54</td>
<td>0.19</td>
</tr>
<tr>
<td>TB-2D-10D-S</td>
<td>Skirted Footing</td>
<td>1.0D</td>
<td>2.16</td>
<td>1.13</td>
<td>4.08</td>
<td>11.47</td>
<td>0.53</td>
<td>0.10</td>
</tr>
<tr>
<td>TB-2D-15D</td>
<td>Flat-base footing</td>
<td>1.5D</td>
<td>0.46</td>
<td>0.26</td>
<td>2.00</td>
<td>3.80</td>
<td>0.16</td>
<td>0.02</td>
</tr>
<tr>
<td>TB-2D-15D-S</td>
<td>Skirted Footing</td>
<td>1.5D</td>
<td>1.99</td>
<td>0.95</td>
<td>2.85</td>
<td>3.68</td>
<td>0.11</td>
<td>0.01</td>
</tr>
</tbody>
</table>

Previous research investigating the use of skirted footings in jack-up reinstallation is relatively limited. The results presented here are compared with Teh et al. (2006), who reported 1g-scaled model test results in heavily over-consolidated clay, and Gaudin et al. (2007), who adopted a free-to-slide jack-up leg and actuator connection. The comparison is presented in Figure 3.44. The scattering of the results is likely to be related to the fundamental difference in the testing condition. The increase in the peak loadings reported here is directly opposite to that reported by Teh et al. (2006). Teh et al. (2006) considered installation of a spudcan and a 0.25D deep skirted footing at $\beta=1.0D$ from a slope. The $(H/\text{As}_u)_{\text{max}}$ is in similar magnitude but in the reverse direction and $(M/\text{As}_uD)_{\text{max}}$ reduced by 30% when a skirted footing is considered. On the other hand, Gaudin et al. (2007) reported 50% increase in $(M/\text{As}_uD)_{\text{max}}$. However, the different in jack-up leg actuator fixity adopted in Gaudin et al. (2007) is likely to have resulted in a significant difference to the horizontal response. The effect of the skirted footing will be investigated further in Chapter 4.
Figure 3.44: Comparison of Maximum Loads from Skirted Footing and Footing Reinstallation with Other Studies

3.7 CONCLUDING REMARKS

This chapter presented results from a comprehensive experimental study on the effect of footprint geometry on the reinstallation of footing and skirted footings. The experimental results were consistent and the peak loadings were found to occur between the touchdown level and the depth of the footprint toe. Critical parameters including the offset distance, footprint size and diameter ratio were identified. These experimental results provide an improved understanding on the isolated geometric effect of footprint to reinstallation response. It also provided a basis for the development of a prediction...
method for the shallow depth reinstallation response, which will be discussed in Chapter 5.
CHAPTER 4. GEOMETRIC EFFECT OF FOOTPRINT – FAILURE MECHANISMS

4.1 INTRODUCTION

As presented in Chapter 3, the jack-up reinstallation response differs according to the footprint geometry. Further visualisation experiments were conducted to better understand the correlation between the footprint geometry and the development of vertical, horizontal and moment loads in a jack-up leg, by penetrating a flat-base footing nearby a manually cut footprint cavity. The main objectives of the experimental investigations were:

- to evaluate the change of soil flow mechanisms with the footprint geometry, including offset distance and footprint size;
- to correlate the identified soil flow mechanism with the development of forces and moments in the footing;
- to identify the combined effect of footprint geometry and soil heterogeneity to the soil flow mechanism and explain the effect to the reinstallation response;
- to evaluate the effect of a skirt on the flow mechanism.

Experiments were conducted in the UWA drum centrifuge on a half-footing model penetrating slightly over-consolidated clay. These tests are referred to as half-footing reinstallation tests, distinguishing them from the full-footing reinstallation tests presented in Chapter 3. Digital images were captured during the full penetration using a digital camera. The particle image velocimetry (PIV) algorithm developed by White et al. (2003) was used to analyse the digital images. This allows the PIV analysis output to be presented as a series of velocity vectors and velocity contours to determine the soil failure mechanism during penetration.

In this chapter, an overview of the experimental techniques is presented, followed by a detailed description on the experimental apparatus, testing procedures and programme. For ease of comparison, the testing condition, including the footprint geometry, the dimensionless soil strength parameter and the model leg actuator conditions replicated that of the experiment presented in Chapter 3. The velocity vector and velocity contours plots obtained from the PIV analysis are compared qualitatively. The following comparisons were conducted to evaluate the effect of various footprint parameters:

- Effect of increase in offset distance from 0.5D to 1.0D to 1.5D for footprint TB;
• Effect of increase in footprint size from footprint TB to the steepest and deepest footprint TC;

• Effect of soil heterogeneity by comparing the reinstallation responses of a footing near footprint TB and to that of a real footprint. Two offset distances 0.5D and 1.0D were considered;

• Effectiveness of mitigation by using a skirted footing. Comparisons were made between reinstallation responses of a skirted footing and a flat-base footing located at 1.0D from footprint TB.

The results are subsequently used in Chapter 5 for the development of a simple method for the prediction of jack-up reinstallation response.

4.2 OVERVIEW OF PIV TECHNIQUES

PIV analysis allows precise quantification of soil flow patterns and distortion zones by comparing pairs of images. The concept of applying the PIV technique to geotechnical problems was first implemented by White et al. (2003). It has subsequently been widely used in qualifying and quantifying soil movements in many geotechnical problems. Example application of PIV techniques in the study of spudcan foundations can be found in Hossain and Randolph (2006), Teh (2007) and Teh et al. (2008).

The GeoPIV8 programme developed by White et al. (2003) was adopted in this study to process the digital images. From each digital image the area of interest is cropped before being divided into interrogation patches, each covering a zone of soil approximately 1mm². Each of these patches was tracked using a cross-correlation algorithm, to identify the movement of that patch of soil between a pair of images, with a measurement precision of 10μm for the field of view used during the experiments. Before processing, each image has been corrected for image distortion arising from the non-coplanarity of the image and object planes, and non-linearity within the image resulting from lens aberrations.

The use of PIV analysis requires a half-symmetrical footing model, to be placed in a rectangular testing box, against a transparent Perspex window (viewing window). During the test the half-footing penetrates into the soil and digital images of the soil movement can be continuously acquired through the viewing window.
4.3 EXPERIMENTAL APPARATUS

The half-footing reinstallation tests were performed in the drum centrifuge facility at UWA. Details of the drum centrifuge have already been discussed in Section 3.3.1. The half-footing models and the setting up of the tests are presented in detail hereafter.

4.3.1 Half-Footing Models

The footings were made from aluminium. Each half-footing model was comprised of a 60mm diameter flat-base footing and a 130mm long leg section. The footing diameter was identical to the model footing adopted in the jack-up reinstallation tests presented in Chapter 3. Figure 4.1 and Figure 4.2 show the as-built half-footing models. To prevent soil or water ingress between the half-footing model and the viewing window of the soil container, a 1 mm diameter o-ring was attached to the face of the footing. The model also featured a stiffening bar, attached to the back of the leg. This was to avoid losing the seal due to the bending of the leg during penetration.

Figure 4.1: Detailed Dimension of the Half-Footing Flat-Base Model (All Dimensions in mm)
4.3.2 Experimental Set-Up

A high resolution (4000 × 3000 pixels) digital camera was placed in front of the testing box to capture images at a rate of 1.5 frame/s (15 frame/mm of footing penetration depth, for footing penetration rate of 0.1mm/s). The camera was mounted on a frame, bolted tightly onto the drum channel. A digital clock, attached to the viewing window aided to identify the time difference between images. To improve the quality of images, a lighting frame comprised of rows of LED lights and a cooling fan were also mounted on the drum channel. The experimental set-up is shown in Figure 4.3 and Figure 4.4.
Figure 4.3: Arrangement of Digital Photography Equipment in the Drum Centrifuge

Figure 4.4: Half-footing Reinstallation Test in Drum Centrifuge – Side View
4.4 EXPERIMENTAL PROCEDURES

4.4.1 Preparation of Soil Samples

The over-consolidated soil sample was firstly consolidated using a consolidation press and then consolidated again under the high gravity environment in the drum centrifuge. A large watertight box, called strongbox, of 360mm × 650mm × 325mm (width × length × depth) was used. A 15mm height coarse sand layer was laid on the bottom of the large strongbox, covered with filter paper, to allow bottom drainage. Drainage holes at the base and sides of the large strongbox allowed water drainage to a standpipe during consolidation. The kaolin slurry was then filled into the large strongbox and was covered by a geo-fabric to allow drainage at the top. The soil sample was consolidated using a consolidation press. The consolidation pressure was applied in stages to the target pressure of 28kPa. When the change in sample height under the final consolidation pressure increment reduced to below 0.1mm/hr, consolidation was considered to have been achieved. The large strongbox containing an over-consolidated soil sample was then removed from the consolidation press.

The over-consolidated soil sample was then carefully cut into 80mm × 257mm × 160mm (width × length × depth) blocks using a blade. Each soil sample block was put into the testing box and covered with geo-fabric (saturated with water) to prevent drying of the soil (Figure 4.5).

The testing box was then fitted into the drum channel and consolidated at 120g under self-weight. Stable pore-pressure was observed after consolidation for 2 days. The centrifuge was stopped and the testing boxes were removed from the drum channel. Each testing box contained an over-consolidated soil sample 120mm deep.
4.4.2 Preparation of Footprints

Footprint types TB and TC were considered in the tests. The dimensions of the idealised footprints are shown in Table 3.6. The testing box was 80mm in width and was too narrow to accommodate idealised footprint with a circular shape. Therefore, the idealised footprint cavity was simplified to a “V” shape slope, with the critical geometric parameters (footprint width $x_F$ and slope angle $z_F$) identical to TB and TC. The simplification is justified as the responses of jack-up reinstallation near an idealised footprint cavity and near a V shape slope were shown to be similar in the full-footing tests of Chapter 3.
The idealised footprint was prepared with a cutting tool similar to the one presented in Section 3.3.6. As shown in Figure 4.7, the cutting blade was mounted on top of the testing box, and was slid across the testing box to remove the soil until the targeted footprint depth ($z_F$) was reached.

Figure 4.7: Preparation of an Idealised Footprint Cavity in the Testing Box
Figure 4.8 shows a testing box ready for half-footing reinstallation tests. Coloured flock was sprinkled on the face of soil sample facing the viewing window in order to provide the necessary contrast to run the subsequent PIV analysis. A viewing window with control markers was installed onto the soil sample. This allowed visual inspection of the soil flow during the penetration test and provided reference to quantify the soil movements in subsequent PIV analysis.

In order to investigate the combined effect of footprint geometry and soil heterogeneity to jack-up reinstallation, real footprints were also considered in this study. The real footprints were created by installation of a half-footing model to $z/D=1.2$; the footing was then extracted immediately simulating an operation time of zero (OT=0). Two tests were considered with footing reinstalling at different offset distance from the real footprints. They were named RE-05D-HF and RE-10D-HF.

In test RE-05D-HF, the half-footing model was reinstalled into the soil at 0.5D offset from the real footprint 45 mins after footprint creation. This simulated an elapsed time (ET) of 10 months between the two installations. However, the coloured flocks under the first installation site were buried within the remoulded soil. This created difficulties in the analysis of soil movement during the reinstallation.

RE-10D-HF was therefore conducted with a different method. After the creation of a real footprint by installation and immediate extraction of the model footing, the drum channel was stopped and the testing box removed. Additional coloured flock was sprinkled on the face of the soil sample facing the viewing window. The testing box was then consolidated in the drum channel again for 2 hours before reinstallation of half-footing model. This corresponded to an ET of 2.2 years between the two
installations. The ET of RE-10D-HF was therefore 2.6 times longer than the ET in RE-05D-HF.

4.4.3 Testing Procedures

The testing box and the digital camera (and other accessories for digital photography) were installed onto the drum channel as shown in Figure 4.4, with the o-ring of the half footing in close contact with the viewing window. The drum centrifuge was then spun up to 100g for at least 3 hours, allowing pore pressure in the soil sample to reach hydrostatic equilibrium. The top of the soil sample was filled with water to maintain the saturation of the sample.

In the tests considering the isolated effect of footprint geometry, the test started with penetration of the half-footing model into the soil sample at a velocity of 0.1 mm/s. During penetration, the digital camera operated at a frame rate of 1.5 frame/s. After reaching the depth of 60 mm below the soil surface ($z=D$), the half-footing model was then extracted from the soil. Upon the completion of the half-footing reinstallation test, the half-footing model was removed and replaced by the t-bar penetrometer. The t-bar was then penetrated in the undisturbed soil (the t-bar test site was at least 1.5D away from the half-footing reinstallation test site) to determine the undrained shear strength profile. The drum centrifuge was then stopped and the testing box was removed from the drum channel for detailed inspection of the failure surface. Another testing box was installed for subsequent half-footing reinstallation tests.

4.5 TESTING PROGRAMME

4.5.1 T-bar Characterisation Tests

A t-bar characterisation test was conducted in each testing box immediately after the completion of the half-footing reinstallation test to confirm the undrained shear strength profile of the soil sample. In addition, one testing box was used exclusively to assess the undrained shear strength profile both in undisturbed soil and within footprint cavity TC. In order to minimise the boundary effect, all t-bar characterisation tests were conducted in the middle of the testing box. Table 4.1 shows the list of t-bar characterisation tests and Figure 4.9 shows the testing box after the completion of the tests.
Table 4.1: List of T-bar Characterisation tests

<table>
<thead>
<tr>
<th>T-bar Characterisation Tests</th>
<th>Distance from Footprint Toe (As a Ratio of Footing Diameter)</th>
<th>Reference Half-Footing Reinstallation Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-1</td>
<td>0.25D</td>
<td></td>
</tr>
<tr>
<td>T-2</td>
<td>1.0D</td>
<td></td>
</tr>
<tr>
<td>T-3</td>
<td>1.0D</td>
<td></td>
</tr>
<tr>
<td>T-4</td>
<td>0D</td>
<td></td>
</tr>
<tr>
<td>T-5</td>
<td>0.5D</td>
<td></td>
</tr>
<tr>
<td>T-6</td>
<td>-</td>
<td>TB-10D-HF</td>
</tr>
<tr>
<td>T-7</td>
<td>-</td>
<td>TB-15D-HF</td>
</tr>
<tr>
<td>T-8</td>
<td>-</td>
<td>TC-10D-HF</td>
</tr>
<tr>
<td>T-9</td>
<td>-</td>
<td>RE-05D-HF</td>
</tr>
<tr>
<td>T-10</td>
<td>-</td>
<td>TB-05D-HF</td>
</tr>
<tr>
<td>T-11</td>
<td>-</td>
<td>RE-10-HF</td>
</tr>
<tr>
<td>T-12</td>
<td>-</td>
<td>TB-10D-S-HF</td>
</tr>
</tbody>
</table>

Figure 4.9: T-bar Characterisation Test Layout in the Testing Box
4.5.2 Half-Footing Reinstallation Tests

Eight half-footing reinstallation tests were performed as presented in Table 4.2. The nomenclature of the tests is in the form of “footprint type-offset distance-remarks-HF”. This follows the labelling of the full-footing reinstallation tests in Chapter 3. The term “HF” was added at the end to distinguish half-footing reinstallation tests. FS, TB, TC and RE refer to flat surface, footprint cavity TB, footprint cavity TC and real footprint. Test FS-HF was conducted by penetrating the flat-base half-footing model into a flat surface of soil. Results were used as a reference for subsequent analysis of the soil flow around the footprint.

Table 4.2: List of Half-Footing Reinstallation Tests

<table>
<thead>
<tr>
<th>Half-Footing Reinstallation Tests</th>
<th>Footprint Type</th>
<th>β</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>FS-HF</td>
<td>Flat Surface</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>TB-05D-HF</td>
<td>B-120mm</td>
<td>0.5D</td>
<td>-</td>
</tr>
<tr>
<td>TB-10D-HF</td>
<td>B-120mm</td>
<td>1.0D</td>
<td>-</td>
</tr>
<tr>
<td>TB-15D-HF</td>
<td>B-120mm</td>
<td>1.5D</td>
<td>-</td>
</tr>
</tbody>
</table>
| TC-12D-HF                        | C-120mm        | 1.2D| • Due to an experimental error, the actual footprint toe level in TC-10D-HF is at z=0.50D, which was shallower than the design footprint toe level of 0.67D  
• To be compared with TB-10D which is with a slightly smaller offset distance (differ by 0.2D) |
| RE-05D-HF                        | Real Footprint-120mm | 0.5D| • Reinstalled footing at 0.5D from HS-HF test site  
• Reinstallation test conducted 45 mins (model time) after footprint creation |
| RE-10D-HF                        | Real Footprint -120mm | 1.0D| • Centrifuge stopped after the footprint creation and the sample was consolidated again for 2 hours (model time) |
| TB-10D-S-HF                      | B-120mm        | 1.5D| • Skirted footing                           |

Figure 4.10 shows the layout and the testing box after the completion of test TB-10D-HF. The testing box was carefully inspected after completion of the half-footing reinstallation tests. The soil deformed radially but was well within the walls of the testing box and the boundaries should have limited effects on the testing results. An example of digital image captured during the test is shown in Figure 4.11.
Chapter 4: Geometric Effect of Footprint – Failure Mechanisms

Figure 4.10: Testing Box after the Completion of Half-Footing Reinstallation Test TB-10D-HF

Figure 4.11: Digital Image Captured During Half-Footing Reinstallation Test at 1.0D from Type B Footprint Cavity (TB-10D-HF)
4.6 EXPERIMENTAL RESULTS

4.6.1 Shear Strength of Soil Sample

The half-footing reinstallation tests were conducted at 100g. Although the testing g level was lower than for the full-footing reinstallation tests (required due to the limitation of the PIV set-up), the dimensionless parameter $kD/s_{um}$ was maintained at about 5 to allow direct comparisons of the results. Figure 4.12 shows the results of t-bar tests conducted in undisturbed soil. The results are in good agreement with the theoretical strength profile (refer to Section 3.5.2 for details). The slight variation between the results are likely to be related to the time difference between tests (as the last t-bar characterisation tests were conducted 14 days after the first one) and natural sample variations. Although the difference increases with penetration depth, a good consistency remains within the depth of $z=0.67D$ (toe depth of footprint TC) relevant to the footing testing. The undrained shear strength profiles may be approximated by a bi-linear expression as follows:

$$s_u = 2.60 + 1.24z, \text{ for } z \leq 1m$$  \hspace{1cm} (4.1)

$$s_u = 2.09 + 1.75z, \text{ for } z > 1m$$  \hspace{1cm} (4.2)

The strength distribution within the footprint cavity TC is shown in Figure 4.13. The undrained shear strength was slightly different to the theoretical strength within 0.5D from the footprint centre (30mm from the footprint toe). However, the agreement with
the theoretical strength improved with increasing depth and distance from the footprint centre.

When the results are compared with the $s_u$ profile of circular idealised footprint TC for the full-footing reinstallation test (Section 3.5.2.1), the agreement between the measured and theoretical $s_u$ from this study is better. The difference is within 1kPa compared to 3kPa in Section 3.5.2.1. The difference is considered to be insignificant. The creation of idealised footprint cavities only induced negligible disturbance to the soil strength and is effective in isolating the footprint geometry effect for the investigation of reinstallation response.

![Contour of Undrained Shear Strength underneath Footprint Cavity TB](image)

**Figure 4.13: Contour of Undrained Shear Strength underneath Footprint Cavity TB**

### 4.6.2 Soil Deformation during Half-Footing Installation Test (FS-HF)

The results of test FS-HIF were first considered to establish a reference for comparison with subsequent analysis of the soil flow around the footprint. The digital images captured at depth $z=0.20D$, $0.35D$ and $1.0D$ from test FS-HF are shown on the left hand side of Figure 4.14. Also shown on the right hand side of Figure 4.14 are digital images from Hossain (2008).

Although Hossain (2008) described results of a spudcan shaped footing, Figure 4.14 shows that the soil failure mechanisms from the two tests were comparable at the three considered depths. According to Hossain (2008), the soil failure mechanism changes at three important penetration depths. The failure mechanism at $z=0.20D$ was shallow.
heave, it changed to on-set of back-flow failure at $z=0.35D$ and finally to fully localised back-flow failure at $z=1.0D$.

The digital images were analysed using the PIV technique to determine the soil movement and the output are presented in normalised velocity vectors and normalised velocity contours. To obtain the velocity vectors and contours, the digital images were analysed in pairs. The footing in the second digital image was 0.67mm deeper than the first. The velocity field of half-footing installation test FS-HF are presented in Figure 4.14, and Figure 4.15.

During relatively shallow penetration ($z/D<0.20$), soil above the footing level moved upward and surface heave was observed. On-set of back-flow failure was observed when penetrating below the critical cavity height ($z/D=0.35$). Instead of moving upwards, the soil flows around the flat-base footing and collapsed on to the top of the flat-base footing. Localised back-flow mechanism was demonstrated with digital images and velocity field plot at the maximum penetration depth of the test ($z/D=1.0$).

In FS-HF, the surface heaves were more confined adjacent to the footing compared to Hossain (2008). For deeper penetration at $z/D=0.35$ and $z/D=1.0$, the flow path around the corners of the flat-base footing was longer than the one observed in the spudcan in Hossain (2008). In FS-HS, the soil had to move around from the base to the top of the footing before collapsing onto the top of the footing.

In general the failure mechanisms obtained from half-footing installation tests are in good agreement with Hossain (2008). The differences were due to the different footing geometry and the soil strength of the soil sample. The normalised soil strength of the soil sample considered in this study was $s_{um}/\gamma'D=5.02$ and $k/\gamma'=0.24$, which was significantly higher than the one adopted in Hossain (2008) ($s_{um}/\gamma'D=0.12$ and $k/\gamma'=0.47$). In Hossain (2008), the footing was penetrated in softer soil and the more extensive soil surface heaves were observed when compared to FS-HF.

The incremental maximum shear strain distributions were also determined from the digital images were analysed using the PIV technique. Figure 4.16 shows that a Hill-type mechanism was observed throughout the footing penetration. A schematic diagram of Hill-type mechanism is shown on Figure A.1. This is typical for a circular footing under undrained loading into non-homogeneous soil (Hill 1951).
Figure 4.14: Digital Images of a Footing Installation into a Flat Surface (LHS: Test FS from This Study; RHS: from Hossain, 2008)
Figure 4.15: Velocity Field of a Footing Installation into a Flat Surface (LHS: Test FS from This Study; RHS: after Hossain, 2008)
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Figure 4.16: Incremental Maximum Shear Strain (LHS) and Velocity Field of Test FS-HS (RHS)

- **Surface Heave Mechanism**
- **On-Set of Back-Flow Mechanism**
- **Fully Localised Back-Flow Mechanism**

Figure 4.16: Incremental Maximum Shear Strain (LHS) and Velocity Field of Test FS-HS (RHS)
4.6.3 Soil Deformation during Half-Footing Reinstallation Tests

A detailed discussion on soil flow mechanism during jack-up reinstallation is now presented based on test TB-10D-HF, which involved reinstallation of a footing at 1.0D offset from footprint cavity TB. As discussed in Section 3.5.4, the response of a footing reinstallation near an idealised footprint cavity exhibited a three-stage response. To capture the change of soil flow during these stages, PIV analysis was conducted at five different depths:

1) “Touchdown” z/D=0.05: Stage 1 response occurred at the touchdown level and also where the maximum moment $M_{\text{max}}$ was recorded.

2) “Full Contact” z/D=0.10: This is approximately mid-way between the stage 1 and 2 response. At this depth the footing came into full contact with the soil (i.e. the contact width is equal to the footing diameter D). This depth is referred to as $z_{\text{contact}}$ and occurred at z/D=0.10 for TB-10D-HF. $z_{\text{contact}}$ varies with footprint geometry and offset distance.

3) “Footprint Toe” z/D=0.33: This is at the end of stage 2 response. This corresponds to the footprint toe level and where the maximum horizontal force $H_{\text{max}}$ is recorded.

4) “Zero H and M” z/D=0.5: Stage 3 response occurred below the footprint toe level. In this stage the maximum moment $M_{\text{max}}$ and horizontal forces $H_{\text{max}}$ reduced to almost zero.

5) “Fully Localised Back-flow” z/D=1.0: The effect of footprint geometry to the reinstallation response ceased when the footing was located more than 1D below the footprint toe level.

In order to determine the effect of change in soil flow on the vertical, horizontal and moment responses of the footing, Figure 4.17 presents the results of full-footing reinstallation test TB-2D-10.
Figure 4.17: Vertical, Horizontal Forces and Moments VHM at the Five Important Depths for Test TB-10D

The digital images of the half-footing at the five important depths are presented in Figure 4.18 to Figure 4.22. In addition, the velocity vectors and velocity contours (normalised by footing penetration velocity) are presented together with the ground profile to identify potential soil heave and back flow.
Figure 4.18: Velocity Vectors and Normalised Velocity Contours at Touchdown Level, z/D=0.05 (TB-10D-HF)
Figure 4.19: Velocity Vectors and Normalised Velocity Contours when Footing Come to Full Contact with Soil, z/D=0.11 (TB-10D-HF)
Figure 4.20: Velocity Vectors and Normalised Velocity Contours at Footprint Toe Level, z/D=0.33 (TB-10D-HF)
Figure 4.21: Velocity Vectors and Normalised Velocity Contours below Footprint Toe Level, $z/D=0.50$ (TB-10D-HF)
Figure 4.22: Velocity Vectors and Normalised Velocity Contours during Deep Penetration, z/D=1.0 (TB-10D-HF)
Based on the velocity fields, the failure mechanisms at different penetration depths are presented in Figure 4.23. The shear planes are shown in red coloured lines. As the movement of the footing is restricted to be purely vertical, the horizontal and eccentric vertical soil reactions translate into vertical, horizontal and moment forces. The eccentric vertical force (green arrows) and the soil movement governing the horizontal force (blue arrows) were also identified from the velocity field in Figure 4.23. For ease of discussion, the side of the footing closer to the footprint centre is referred to as LHS and the other side is referred to as RHS. The definitions of terminologies and the simplified failure mechanisms are shown in Figure 4.24.

In stage 1 \((z/D=0.05)\), a two-way mechanism was observed on the partially supported footing. The observed failure was similar to the classic Hill-type mechanisms, but the point of separation of the two-ways mechanism (indicated by the green arrow in Figure 4.18) is at a distance \(x_{\text{neutral}}\) from the centreline of the footing (see Figure 4.24).

The passive wedge on the LHS is displaced horizontally towards the footprint centre. This is the optimal mechanism as the soil is following the path of less resistance with shear forces along the failure slip involved in the process. In contrast, the soil deformation on the RHS exhibits a typical Hill-type mechanism. The two-way mechanism generates a penetration resistance, which is indeed lower than the one generated by a full bearing failure mechanism, as demonstrated in Figure 4.17. The dissymmetry of the mechanism also generates an eccentricity of the resultant of the vertical stresses at the footing invert, which tends to rotate the footing away from the footprint centre. This translates into a moment on the footing leg, as the footing is prevented from rotating. As evident in Figure 4.17, the maximum moment \(M_{\text{max}}\) occurred at the touchdown level. This is where the dissymmetry of the mechanism is the most pronounced and consequently where the eccentricity is at a maximum. The footing sitting on the footprint also tends to move horizontally with the failed soil moving towards the footprint centre. Again, the movement was transmitted to a horizontal force because the footing was restricted from movement. As shown in TB-2D-10D, the horizontal force increased gradually from the touchdown level.

As penetration continued to stage 2, the footing came into full contact with the founding soil at \(z_{\text{contact}}\ (z/D=0.10)\) and a two-ways mechanism is still observed. However, the shear plane develops along a longer length, hence generating a higher resistance. It results in (i) a penetration resistance still lower than the penetration resistance of test FS, but to a lesser extent, and (ii) a reduction of the eccentricity of the resultant of the vertical stresses at the footing invert. The reduction of the eccentricity results, however, in a marginal reduction of the moment in the footing leg, because it is compensated by the increase of vertical resistance associated with the increasing contact area between the footing and the soil. The size of the horizontal moving soil block also increased with penetration until reaching \(z_{\text{contact}}\). This imposed a horizontal force on the footing sitting on top of the soil and the footprint therefore experienced an increase in horizontal
force with increasing penetration depth. In addition, the RHS of the footing starts to be embedded into the soil and there was unbalanced lateral earth pressure acting on the two sides of the footing. This unbalanced earth lateral pressure contributed to the increase in horizontal force with penetration depth until the soil started to flow from the bottom around the footing.

At footprint toe level \((z/D=0.33)\), the soil velocity field exhibits a more symmetrical pattern. As a consequence, the penetration resistance reaches values closer to test FS (see Figure 4.18), and the eccentricity of the vertical forces at the footing invert and the resulting moment reduces to almost zero. An open cavity was formed on the LHS of the footing. A Hill-type mechanism is observed on the LHS of the footing while full flow failure starts to occur on the RHS. Although the passive wedge moves upward to ground surface, rather than horizontally towards the footprint toe, the horizontal force continues to increase due to the different overburden soil stress on the two sides of the footing and induces an unbalanced lateral earth pressure on the footing.

At \(z/D=0.50\) the footing reaches stage 3 penetration response and a full flow mechanism occurs on both side of the footing. It results in a moment reducing to almost zero and to the relatively small horizontal force shown in TB-2D-10D. For deeper penetration at \(z/D=1.0\), the soil flow is perfectly symmetrical and the influence of the idealised footprint cavity is no longer noted.

In conclusion, the footing initially failed in a two-way mechanism and then the failure mechanism changed to a skewed bearing failure near \(z_{\text{contact}}\). With further penetration below \(z_f\), the mechanism changed to a one-side full flow failure until the effect of the footprint ceased. The point of separation of the two-way failure, \(x_{\text{neutral}}\) shifted closer to the footing centreline. The change of failure mechanisms with increasing penetration depth is summarised in Figure 4.24. The footing experienced eccentric vertical force and therefore moment in the footing. Furthermore, the combined effect of the failed soil moving horizontally and the different overburden stress on the two side of footing resulted in horizontal force acting on the footing.
Figure 4.23: Change of Failure Mechanisms with Reinstallation Depth (TB-10D-HF)
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4.6.4 Summary of Experimental Results

PIV analysis has been conducted on all half-footing reinstallation tests and the depths selected for analysis are shown in Table 4.3. The results are presented in subsequent sections, focusing on the influence of the offset distance, the footprint size and the soil heterogeneity. The failure mechanisms of skirted footing reinstallation were also considered.

Figure 4.24: Typical Failure Modes of Footing Reinstallation Near Footprint
### Table 4.3: Summary of PIV Analysis for Half-Footing Reinstallation Tests

<table>
<thead>
<tr>
<th>Test Name</th>
<th>Footprint Type</th>
<th>β</th>
<th>Touch-down</th>
<th>z_{contact}</th>
<th>z_F</th>
<th>Zero H and M</th>
<th>Full Back-flow</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>TB-05D-HF</td>
<td>TB</td>
<td>0.5D</td>
<td>0.05</td>
<td>0.13</td>
<td>0.33</td>
<td>0.5</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>TB-10D-HF</td>
<td>TB</td>
<td>1.0D</td>
<td>0.05</td>
<td>0.10</td>
<td>0.33</td>
<td>0.5</td>
<td>1.0</td>
<td>-</td>
</tr>
<tr>
<td>TB-15D-HF</td>
<td>TB</td>
<td>1.5D</td>
<td>0.05</td>
<td>-</td>
<td>0.33</td>
<td>0.5</td>
<td>-</td>
<td>0.10</td>
</tr>
<tr>
<td>TC-12D-HF</td>
<td>TC</td>
<td>1.2D</td>
<td>0.05</td>
<td>0.10</td>
<td>0.50*</td>
<td>-</td>
<td>-</td>
<td>0.33</td>
</tr>
<tr>
<td>RE-05D-HF</td>
<td>Real Footprint</td>
<td>0.5D</td>
<td>0.05</td>
<td>0.10</td>
<td>0.33*</td>
<td>0.5</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>RE-10D-HF</td>
<td>Real Footprint</td>
<td>1.0D</td>
<td>0.05</td>
<td>0.13</td>
<td>0.33*</td>
<td>0.5</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>TB-10D-S-HF</td>
<td>TB</td>
<td>1.0D</td>
<td>0.05</td>
<td>0.08</td>
<td>0.33</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

*Actual footprint toe level

*Approximated toe level of real footprint

### 4.7 COMPARISON AND DISCUSSION OF EXPERIMENTAL RESULTS

In this section, the PIV analysis results are compared with the full-footing reinstallation test results presented in Chapter 3. Discussions are concentrated on correlating the observed soil flow mechanism with the occurrence of H_{max} and M_{max}. The comparisons and discussions provide insights to the fundamental soil mechanisms governing the reinstallation response.

#### 4.7.1 Effect of Offset Distance

Figure 4.25 shows the results from full-footing reinstallation tests (refer to Chapter 3 for details) confirming that the horizontal force and moment were highest for reinstallation at an offset distance of 1.0D (β=1.0D). There is however no existing correlation or understanding about how the change of soil flow during footing penetration at different offset distance might influence the penetration response. Therefore, half-footing reinstallation tests were conducted at β=0.5D, 1.0D and 1.5D to investigate the change of soil flow with offset distance. The PIV analysis results are shown in Figure 4.26 to Figure 4.29 which presents the normalised velocity vectors and velocity contours at different stages of penetration.
At touchdown level $z/D=0.05$, although the same two-way mechanism was observed in all three tests, the size of horizontal moving soil block became more significant with reducing offset distance. The eccentricity of the vertical forces at the footing invert is the lowest for test TB-15D-HF, resulting in the lowest $M_{\text{max}}$. The contact area in test TB-05D-HF is the smallest, resulting in the lowest vertical force. Consequently, though the eccentricity in test TB-05D-HF is the greatest, the vertical force results in only a relatively small moment. As penetration continued, the failed soil filled the void underneath the footing and the footing came into full contact with the soil at $z_{\text{contact}}=0.13D$ and $0.10D$ in tests TB-05D-HF and TB-10D-HF, respectively. The vertical force increased significantly with increasing contact area. The vertical force in TB-05D-HF remained highly eccentric and gave the largest $M_{\text{max}}$ among the three tests. In the other two tests, TB-10D-HF and TB-15D-HF, the moment at $z_{\text{contact}}$ fluctuated around $M_{\text{max}}$ because the vertical resistance remained highly eccentric. The vertical forces became symmetrical near footprint toe level at $z/D=0.33$. Therefore, the moment reduced to almost zero in all three tests.
Figure 4.26: Velocity Vectors and Normalised Velocity Contours of Footing Reinstalled at Different Offset Distance From Footprint Type B (z/D=0.05)
Figure 4.27: Velocity Vectors and Normalised Velocity Contours of Footing Reinstalled at Different Offset Distance From Footprint Type B (z=z_{contact})
Figure 4.28: Velocity Vectors and Normalised Velocity Contours of Footing Reinstalled at Different Offset Distance From Footprint Type B (z/D=0.33)
Figure 4.29: Velocity Vectors and Normalised Velocity Contours of Footing Reinstalled at Different Offset Distance From Footprint Type B (z/D=0.50)
The PIV analysis results indicate that the size of the horizontal moving soil block in TB-10-HF was the largest of the three tests. It imposed a larger horizontal force on the footing. The footing in the three tests was embedded in soil of different heights during reinstallation. The unbalanced lateral earth pressures on the two sides of the footing also contributed to the horizontal force acting on the footing. The unbalanced lateral earth pressures increased with reducing offset distance. The \( H_{\text{max}} \) in test TB-10D-HF is the largest of the three tests because it has the largest horizontal moving soil block and modest differences in lateral earth pressure on the two sides of footing. In TB-10D-HF, the horizontal force continued to increase from touchdown level until the cavity on the RHS started to collapse below the footprint toe level \((z/D=0.33)\). \( H_{\text{max}} \) occurred near the footprint toe level \((z/D=0.33)\). The horizontal force then decreased, and at \( z/D=0.5 \) reached zero, as the dissymmetry in the soil back-flow mechanism disappeared.

For a detailed exploration of the change in failure mechanisms with depth, the soil movement in the horizontal direction, \( du \), was evaluated at each depth of PIV analyses for quantitative comparison. Figure 4.30 shows \( du \) of the footing at touchdown level (of test TB-10D-HF). At the point of separation, the soil movement is predominately vertical as the horizontal displacement equals to zero \((du=0)\). The soils on the two sides of the point of separation move in reversed direction. This point is the point of separation of the two-ways mechanism and is defined as \( x_{\text{neutral}} \).

![Figure 4.30: The Horizontal Movement at the Base of the Footing (TB-10D-HF at z=0.05)](image)

The change of normalised \( x_{\text{neutral}} \) with depth for footing reinstallation at different offset distances are summarised in Table 4.4. At touchdown level, \( x_{\text{neutral}}/D \) was largest for an offset distance of 0.5D \((x_{\text{neutral}}/D=0.43)\), indicating that the eccentricity was the greatest.
For penetration near footprint toe level $z_F$, the $x_{\text{neutral}}/D$ was almost zero for all offset distances. This is consistent with the relatively low moment observed at $z_F$ from the full-footing reinstallation tests. For TB-15D-HF, the point of separation was the footing centreline throughout the reinstallation. This gave TB-15D-HF the lowest moment among the three tests.

<table>
<thead>
<tr>
<th>Table 4.4: Point of Separation of the Two-Way Failure Mechanism</th>
</tr>
</thead>
<tbody>
<tr>
<td>$z/D$</td>
</tr>
<tr>
<td>-------</td>
</tr>
<tr>
<td>0.05</td>
</tr>
<tr>
<td>0.10</td>
</tr>
<tr>
<td>0.33</td>
</tr>
<tr>
<td>0.50</td>
</tr>
</tbody>
</table>

### 4.7.2 Effect of Footprint Size

Figure 4.31 shows that footings installed near the deepest and steepest footprint TC experienced a $H_{\text{max}}$ two times and a $M_{\text{max}}$ three times higher than footings installed near footprint TB. PIV analysis results on reinstallation tests near the two different footprint types were therefore compared to identify the effect of footprint size on failure mechanisms. Although the offset distance in TC-12D-HF was 0.2D larger than TB-10D-HF, the difference in the failure mechanisms would be dominated by the change of footprint type, rather than by the relatively small difference in the offset distance. Comparisons should therefore be relevant to the investigation of the effect of footprint size.
Figure 4.31: Vertical, Horizontal Forces and Moments of Footing at $\beta=1.0D$ from Footprints TB and TC

Figure 4.32 to Figure 4.35 compare the soil flow mechanisms from the two tests. Footprint TC is with steeper slope angle and deeper slope height. Therefore, it has a lower bearing resistance compared to footprint TB. The vertical force in TC-10-HF was lower than in TB-10-HF. However, the eccentricity in TC-10D-HF was also the highest. Therefore, the moment at touchdown level for reinstallation near footprint TB and TC were similar. The two-ways mechanism in the two tests gradually changed to skewed bearing failure with further penetration. The vertical force in TC-10D-HF remained to be highly eccentric and the moment continued to increase and reached $M_{\text{max}}$ at $z_{\text{contact}}$. The more eccentric vertical force in footprint TC gave a larger $M_{\text{max}}$ compared to the case of TB.

The larger horizontal moving soil block in TC-10D-HF gave a larger horizontal force near the touchdown level and the $H_{\text{max}}$ occurred near $z_{\text{contact}}$ instead of near footprint toe level as in TB-10D-HF. The deeper footprint slope in TC-10D-HF also resulted in a higher unbalanced lateral earth pressure. The soil on the RHS side of the footing continued to build up generating an unbalanced lateral earth pressure until the cavity collapses at about $z/D=0.33$. Combining the larger horizontal moving soil block and the larger unbalanced lateral earth pressure, the $H_{\text{max}}$ in TC was therefore larger than in TB.
Figure 4.32: Velocity Vectors and Normalised Velocity Contours of a Footing Reinstalled near Footprint TB and TC (z/D=0.05)
Figure 4.33: Velocity Vectors and Normalised Velocity Contours of a Footing Reinstalled near Footprint TB and TC \( z=z_{\text{contact}} \)
Figure 4.34: Velocity Vectors and Normalised Velocity Contours of a Footing Reinstalled near Footprint TB and TC (z/D=0.33)
4.7.3 Effect of Soil Heterogeneity

The tests presented in the previous sections considered the isolated effect of footprint geometry. The combined effect of footprint geometry and soil heterogeneity was also investigated by the half-footing reinstallation tests at offset distance of 0.5D and 1.0D from real footprints. The results of real footprint tests RE-05D-HF and RE-10D-HF were compared with the equivalent idealised footprint cavity tests TB-05D-HF and TB-10D-HF, respectively. The recorded ground profiles of the real footprints are also compared with footprint cavity TB in Figure 4.36. Although the footprint toe level of footprint cavity TB was deeper than the real footprints, its diameter is comparable with the profiles of real footprints. The footprint cavity TB therefore provided a good approximation of the geometry of real footprints.
The soil strength contours from Gan (2009)’s experimental investigation will be used in the following discussion on the combined effect of footprint geometry and soil heterogeneity. Gan (2009)’s soil strength contours are presented in Figure 4.37 and Figure 4.38. Since the elapsed time (ET) adopted in RE-10D-HF were 2.6 time longer than in RE-05D-HF, the test results will be discussed separately.
The reinstallation response of footing at an offset distance of 0.5D is shown in Figure 4.39 to Figure 4.42. Since the real footprint has a highly irregular ground profile, at touchdown level the contact area in RE-05D-HF was slightly larger than the one in TB-05D-HF. As penetration continued from stage 1 to stage 2 (0.05<z/D<0.33), a two-way failure mechanism was observed in RE-05D-HF, which is similar to TB-05D-HF.
Figure 4.37 shows that for a footing located at 0.5D offset from the footprint centre, although the soil strength around the real footprint was lower than in the undisturbed soil, $s_u$ was rather uniform. The eccentricity of vertical resistance was therefore controlled by the footprint geometry. For depth below the footprint toe level ($z/D > 0.33$), the bearing failure in RE-05D-HF was only slightly skewed. This again is related to the rather uniform $s_u$ for footing reinstalling at 0.5D from the footprint centre. Although the eccentricity of vertical resistance were similar between RE-05D-HF and TB-05D-HF, the softer soil around the real footprint should give lower vertical resistance and is likely to induce a lower $M_{\text{max}}$ during reinstallation near a real footprint than in a idealised footprint cavity.

The size of horizontal moving soil block in the real footprint and in an idealised footprint cavity were comparable and should therefore have a similar contribution to the horizontal force acting on the footing. However, in the case of real footprint, the soil heaved up quickly on the LHS and reduced the difference in the lateral earth pressure on the two sides of the footing. It is because the soil closer to the centre of real footprint has lower soil strength. The unbalanced lateral earth pressure acting on the footing reinstalling in a real footprint was therefore less significant than in the idealised footprint cavity. As a result the $H_{\text{max}}$ in RE-05D-HF is likely to be lower than the in TB-05D-HF.
Figure 4.39: Velocity Vectors and Normalised Velocity Contours of a Footing Reinstalled at 0.5D from a Real Footprint and an Idealised Footprint Cavity TB ($z/D=0.05$)
Figure 4.40: Velocity Vectors and Normalised Velocity Contours of a Footing Reinstalled at 0.5D from a Real Footprint and an Idealised Footprint Cavity TB ($Z_{contact}$)
Figure 4.41: Velocity Vectors and Normalised Velocity Contours of a Footing Reinstalled at 0.5D from a Real Footprint and an Idealised Footprint Cavity TB (z/D=0.33)
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The results of footing reinstallation at $\beta=1.0D$ are presented in Figure 4.43 to Figure 4.46. At touchdown level, the horizontal moving soil block in RE-10D-HF was larger and deeper than the one in TB-10D-HF. Figure 4.38 shows relatively large $s_u$ variation underneath the footing located at $\beta=1.0D$ from the real footprint. This contributed to the formation of a large failure zone. With further penetration, this relatively large and deep horizontal moving soil block was observed throughout stage 1 and 2 reinstallation. This failure mechanism was also observed in Gan (2009) and was referred to as a sliding mechanism (Figure 4.47). The dissymmetry reduced slightly with further penetration but the vertical resistance remained eccentric throughout the penetration.

The ET in RE-10D-HF was 2.6 times longer than in RE-05D-HF. Therefore, the remoulded soil around the real footprint in RE-10D-HF had a longer time to consolidate and regain some strength. The effect of soil heterogeneity in RE-10D-HF should,
therefore, be less significant than in RE-05-HF. However, the vertical resistance in RE-10D-HF was still significantly more eccentric than in RE-05D-HF. This confirmed Gan (2009) and Cassidy et al. (2009)’s conclusions that the moment responses during reinstallation near a footprint is most critical at $\beta=1.0D$.

The full-reinstallation tests presented in Chapter 3 only considered the idealised footprint cavity. Therefore, comparison on the vertical, horizontal and moment loads during reinstallation near a real footprint and an idealised footprint cavity cannot be undertaken. However, based on the above observation from the PIV analyses, instead of reducing to zero near the footprint toe level (the case for reinstallation near idealised footprint cavity), the horizontal force and moment should remain non-zero during reinstallation near the real footprint. This is in agreement with the observation from Gan (2009).

\begin{figure}
\centering
\includegraphics[width=\textwidth]{figure4.43}
\caption{Velocity Vectors and Normalised Velocity Contours of a Footing Reinstalled at 1.0D from a Real Footprint and an Idealised Footprint Cavity TB ($z/D=0.05$)}
\end{figure}
Figure 4.44: Velocity Vectors and Normalised Velocity Contours of a Footing Reinstalled at 1.0D from a Real Footprint and an Idealised Footprint Cavity TB (z=z_{contact})
Figure 4.45: Velocity Vectors and Normalised Velocity Contours of a Footing Reinstalled at 1.0D from a Real Footprint and an Idealised Footprint Cavity TB (z/D=0.33)
Figure 4.46: Velocity Vectors and Normalised Velocity Contours of a Footing Reinstalled at 1.0D from a Real Footprint and an Idealised Footprint Cavity TB (z/D=0.50)

Figure 4.47: Possible Failure Mechanism during Half-Footing Reinstallation near a Real Footprint as Suggested by Gan (2009)
4.7.4 Effect of Skirted Footing

Skirted footings have been considered by industry as a potential mitigation measure for problems during a jack-up reinstallation next to a footprint (Teh et al. 2006, Gaudin et al., 2007). However, full-footing reinstallation tests results shown in Section 3.6.7 and Figure 4.48 indicated that $M_{\text{max}}$ and $H_{\text{max}}$ acting on the skirted footing were two times and three times larger respectively than that of the flat-base footing for a fully fixed jack-up. Test TB-10D-HF-S was conducted and compared with TB-10D-HF to better understand the mechanism governing the increase in loading with the use of skirted footing. The velocity fields of test TB-10D-S-HF are presented in Figure 4.49 to Figure 4.51. The corresponding full-footing reinstallation test result TB-2D-10D-S is shown in Figure 4.48.

![Figure 4.48: Vertical, Horizontal Forces and Moments of a Flat-Base Footing (TB-2D-10D) and a Skirted Footing (TB-2D-10D-S) at $\beta=1.0D$ from Footprint TB](image)

It is evident from Figure 4.49 to Figure 4.51 that the presence of the skirt modifies significantly the failure mechanism during reinstallation. During reinstallation near touchdown level, a relatively small horizontal moving soil block was observed beneath the top plate of the skirted footing. The skirt on the RHS was embedded into the soil transferring the load to stronger soil at greater depth. The failure region was extended to below the skirt tip level. This transferred the loading to stronger soil at greater depth, together with the additional shaft resistance from the skirt give a higher vertical resistance in a skirted footing than a flat-base footing. However, the moment at touchdown level was similar between the skirted footing and the flat-base footing. A possible reason is that the vertical resistance from the deep seated failure was less.
eccentric than the flat-footing test, and this offsets the effect from the resistance on the RHS of the skirted footing.

As the top plate of the skirted footing came into full contact with the soil, the skirt on the two sides of the footing were both embedded in the soil. The loading was transferred to the skirt tip level and the deep seated failure mechanism was symmetric and the moment reduced significantly.

In addition, the soil within the skirted footing was restrained and could not move towards the footprint toe. This induced a horizontal force on the skirted footing at shallow depth. The skirted footing also experienced a larger unbalanced lateral earth pressure. The horizontal forces in the skirted footing therefore increased at a larger rate with penetration depth than in flat-base footing. $H_{\text{max}}$ in both tests occur at approximately footprint toe level ($z=0.33D$) and $H_{\text{max}}$ in TB-2D-10D-S was almost three times larger than TB-2D-10D. The large horizontal force also induced a wedge failure on the LHS of the skirted footing.

Unfortunately, due to the loss of seal between the half-footing model and the viewing window, the soil did not move together with the footing for penetration below $z_F$. The image quality was not sufficient to conduct PIV analysis below $z_F$. 
Figure 4.49: Velocity Vectors and Normalised Velocity Contours of a Flat-base Footing and a Skirted Footing at 1.0D from Footprint TB (z/D=0.05)
Figure 4.50: Velocity Vectors and Normalised Velocity Contours of a Flat-base Footing and a Skirted Footing at 1.0D from Footprint TB ($z=z_{contact}$)
4.8 CONCLUDING REMARKS

In this chapter, failure mechanisms during jack-up reinstallation have been investigated qualitatively using PIV techniques. The key findings are summarised below:

- A two-way mechanism was observed during stage 1 ($z<0.05D$) and stage 2 ($0.05D<z<z_F$) penetration of footing reinstallation near an idealised footprint cavity and the failure mechanism gradually changed to a skewed bearing mechanism with further penetration;

- PIV analysis reveals that the eccentric vertical resistance controlled the moment response throughout the penetration, while the horizontal response was dominated by the combined force of a horizontal moving soil block and unbalanced lateral earth pressure;
• Critical moment responses occurred at $\beta=1.0$ because vertical resistance was modest but highly eccentric. Critical horizontal force occurred at $\beta=1.0$ because the horizontal moving soil block was largest and the unbalanced lateral earth pressure was modest.

• The deeper footprint TC gives a deeper and larger horizontal moving soil block and larger unbalanced lateral earth pressure. This resulted in $H_{\text{max}}$ and $M_{\text{max}}$ in the deeper TC footprint two times and three times larger respectively than in the moderate TB footprint (for $\beta=1.0$).

• The inclusion of soil heterogeneity did not change the failure mechanism. The same two-way mechanism was observed in reinstallation near an idealised footprint cavity and a real footprint. The observed failure mechanism was in good agreement with the one reported by Gan (2009).

• The skirted footing exhibited a combined shallow bearing failure and deep seated failure mechanisms at the touchdown level during reinstallation near an idealised footprint cavity.

• The larger unbalanced lateral earth pressure acting on the two sides of the skirted footing gave a $H_{\text{max}}$ three times larger than the case of flat-base footing.

The failure mechanisms inferred from half-footing reinstallation tests and PIV analyses provided useful insights to change of reinstallation response with footprint geometries (in terms of offset distance and footprint size) and soil heterogeneity. The observation from the test results also formed the basis for the development of a prediction method in Chapter 5.
CHAPTER 5. PREDICTION OF JACK-UP REINSTALLATION RESPONSE

5.1 INTRODUCTION

For safe jack-up reinstallation, it is necessary to ensure the loading does not exceed the capacity of the jack-up unit, otherwise damage to the structure and even collapse of the jack-up unit could occur. The current design guideline SNAME (2002) only provides general advice about keeping a minimum distance of one footing diameter between the edge of the footing and the edge of the footprint. However, such a minimum offset distance might not be achievable, as the jack-up might have to be reinstalled closer to the footprint to allow work over an existing platform. In such cases it would be desirable to predict the reinstallation response allowing the selection of an appropriate jack-up unit with sufficient capacity.

Prediction of the reinstallation response for offshore cases is possible using physical modelling or large deformation finite element modelling. However, as both require significant resources, expertise and time, a simple predictive method to allow industry a “first-pass” estimate of the foundation loading is desirable. This is one of the key objectives of this study and the topic of this chapter. In this chapter, assumptions to the simplified jack-up reinstallation problem are first presented. The key parameters are then back-calculated from the results of full-footing and half-footing reinstallation tests (detailed in Chapters 3 & 4). Finally, the prediction method is validated by comparing it with two other experimental studies of jack-up reinstallation near real footprints. The performance of the simple prediction method is discussed at the end of the chapter.

5.2 THE SIMPLIFIED JACK-UP REINSTALLATION PROBLEM

Gan (2009) presented a simple method for initial estimation of the peak responses and this is the only published prediction method. However, it was subjected to the limitation that it was developed based on $\beta=0.5D$ and only considered a relatively short operation period and elapsed time. The effects of changing the footprint geometry (or offset distance) and soil heterogeneity (influence of operation time and elapsed time) on the prediction accuracy are unclear.

To accurately predict the reinstallation response, all three key governing parameters—the footprint geometry, the soil heterogeneity and the jack-up structural stiffness—must be accounted for in the prediction method. However, such a method would be complex to develop and would require a relatively large amount of input data. The jack-up
reinstallation problem was therefore simplified, and the three key assumptions adopted in the simple prediction method are listed below:

1) The method only considers the effect of the footprint geometry and predicts the shallow depth response. As already presented in Chapter 3 and 4 the shallow depth horizontal force and moment response were primarily controlled by the footprint geometry and the offset distance. The deep reinstallation response that was governed by the soil heterogeneity was not considered in the method.

2) The geometries of idealised footprint cavities were simplified into the three conical footprint types presented in Chapter 3. These footprint types were developed based on the findings from a literature review and centrifuge testing.

3) The jack-up was assumed to have infinite stiffness at the footing level. The predicted horizontal force and moment are therefore conservatively higher than they are for the finite structural stiffness case found offshore, where the reinstalling footing has potential to slide and/or rotate towards the footprint.

### 5.3 OVERVIEW OF PREDICTION METHOD

The prediction method requires two key input parameters: 1) the undrained shear strength of the undisturbed soil and 2) the footprint geometry. The procedures of the prediction method are outlined in Figure 5.1.

The vertical load is predicted by modifying the equation for full bearing capacity with a reduction factor, RF. RF accounts for both the reduction in contact area and the lower soil resistance from the geometry of footprint. To predict the moment, the maximum eccentricity and the trend of change in eccentricity with reinstallation depth are firstly estimated from design charts. The moment can then be predicted from the product of the eccentricity and predicted vertical load. Similarly, the horizontal force is predicted by firstly estimating the maximum inclination angle and the trend of change in inclination with reinstallation depth. The horizontal force can then be predicted from the product of the inclination angle and predicted vertical load. The maximum values and the trend of change in eccentricity and inclination angles were calibrated from centrifuge test data and numerical modelling results. The development of the simple prediction method is presented in detail in this chapter.
Chapter 5: Prediction of Jack-Up Reinstallation Response

**Figure 5.1: Procedure of Simple Prediction Method**
5.4 PREDICTION OF VERTICAL FORCE

In this section, two prediction methods: Houlsby and Martin (2003) and Hossain and Randolph (2009) are firstly compared with centrifuge test data. These two methods calculate the vertical responses of spudcan penetration into a flat undisturbed surface. The method developed by Hossain and Randolph (2009) was then modified to estimate the vertical force during reinstallation.

5.4.1 Vertical Force from the Full-Footing Installation Test in Flat Surface

The full-footing installation test results (on a flat surface) presented in Chapter 3 are compared with Houlsby and Martin (2003) and Hossain and Randolph (2009) developed specifically for spudcan foundations. A detailed review of the two prediction methods is presented in Appendix A. The comparisons of the jack-up installation test results with the two prediction methods are shown in Figure 5.2 and Figure 5.3. Because the footing is neither fully smooth nor rough in reality, both cases were considered. It was found that Houlsby and Martin (2003) under-estimated the vertical bearing capacity throughout the penetration depth. Although based on a conical footing, the vertical capacity predicted using Hossain and Randolph (2009) was consistent with the test results obtained from flat-base footings.

There was slight over-prediction at z/D between 0.33 to 0.6, which was at the transition of the failure mechanism from cavity formation to localised flow failure. However, this mechanism was relatively complex, and the 15% difference between the measured and predicted Nc values was considered to be acceptable.
Figure 5.2: Comparison of Results for Jack-up Installation on a Flat Surface with Published Spudcan Penetration Theories

Figure 5.3: Comparison of the Bearing Capacity Factor for Jack-up Installation on a Flat Surface with Published Spudcan Penetration Theories
5.4.2 Vertical Force from Full-Footing Reinstallation Test near Idealised Footprint Cavities

As revealed from a detailed literature review, there is no existing method for the prediction of the vertical capacity of a circular footing near a sloping ground in non-homogeneous clay soil. The details of the literature review are presented in Appendix B. To improve the understanding of the vertical responses during reinstallation, the full-footing reinstallation test results were interpreted in detailed in this section.

During jack-up reinstallation, the vertical capacity is lower than the initial installation because of the reduction in the footing-soil contact area and the lower soil resistance from the geometry of the seabed depression. These two effects are quantified by considering the reduced bearing capacity factor, $N_{RF}$, which is defined in Equation (3.1):

$$N_{RF} = \frac{V - \gamma' A_{\text{Full}} s_u}{A_{\text{Full}} s_u}, z < H_s$$

$$N_{RF} = \frac{V - \gamma' A_{\text{Full}} H_s + \left(\frac{\gamma' A_{\text{Full}} H_s - \gamma' V_{\text{Vol}}}{0.3D}\right)(z - H_s)}{A_{\text{Full}} s_u}, H_s < z < H_s + 0.3D$$

$$N_{RF} = \frac{V - \gamma' V_{\text{Vol}}}{A_{\text{Full}} s_u}, z > H_s + 0.3D$$

where $s_u$ is the undrained shear strength of the undisturbed soil and $V$ is the vertical capacity of the footing at each depth of the reinstallation. $H_s$ is the critical cavity depth as defined in Hossain and Randolph (2009). It was assumed the soil completely filled up the cavity at depth of 0.3D below $H_s$. $V_{\text{Vol}}$ and $A_{\text{Full}}$ are the full volume and full area of the footing and are the same as the flat surface case. Because both $N_{RF}$ and $N_c$ (for FS) are based on the full-footing area and volume, any difference between the $N_{RF}$ and $N_c$ reflects the combined effect of the reduction in the footing-soil contact area and the reduction in the soil resistance from the seabed depression. $N_{RF}$ could be determined without considering the complex change in the footing contact area and the embedded volume with penetration depth. However, this makes $N_{RF}$ different from the standard bearing capacity definition, as it is no longer based on the actual foundation contact area and embedded volume.

$N_{RF}$ values back-analysed from full-footing reinstallation tests are shown in Figure 5.4. The tests involved the penetration of a circular footing into non-homogenous soil adjacent to an idealised footprint cavity. The testing details are presented in Chapter 3. In Figure 5.4, $N_c$ values of the same footing on a flat surface calculated from Hossain and Randolph (2009) are also shown for ease of comparison.
The differences between $N_c$ and $N_{RF}$ were largest near the touchdown level because of the relatively small footing-soil contact area and the low soil resistance from the intact footprint slope. As penetration continued, the footing came into full contact with the founding soil, and the footprint slope also deformed and reduced its slope height and angle. The difference between $N_c$ and $N_{RF}$ therefore gradually reduced with penetration depth.

$N_{RF}$ generally increased with offset distance $E$ and $N_{RF}$ for reinstallation at different offset distances converged near the footprint toe level ($z_F$). $N_{RF}$ also reduced significantly as the footprint deepened ($TA \rightarrow TB \rightarrow TC$). In TC-2D-05D, $N_{RF}$ was zero during shallow depth reinstallation ($0 < z/D \leq 0.3$). This occurred because the measured vertical load was relatively low compared with the relatively large capacity from the open cavity ($V \leq \gamma' A_{Full} z$). $N_{RF}$ therefore falls into a negative range. In order to avoid confusion with negative bearing capacities, the value was therefore taken as zero.

The difference between $N_c$ and $N_{RF}$ at a $z/D$ between 0.33 to 0.6 was likely to be related to the slight over-estimation of the Hossain and Randolph (2009) method (refer to Section 5.4.1 for details).

![Figure 5.4: $N_{RF}$ from Full-Footing Reinstallation Tests](image)

The effect of the footprint was further investigated by considering the ratio of $N_{RF}$ to $N_c$, called RF. RF values were back-analysed from full-footing reinstallation tests and are shown in Figure 5.5 and Figure 5.6. RF values for footings at different $E$ varied greatly near the touchdown level, but the difference decreased near $z_F$. It could therefore be confirmed that RF is a function of $z$, $E$ and footing geometries.
5.4.3 Introduction to Vertical Force Predictive Method

Because the vertical capacity of the footing near the idealised footprint cavity is related to the soil strength and the footprint geometric parameters, it is therefore difficult to determine from a theoretical solution. A semi-empirical prediction method is presented here. The comparison given in earlier sections demonstrated that the vertical forces predicted using Hossain and Randolph (2009) were in good agreement with the centrifuge test results. Therefore, Hossain and Randolph (2009) was modified for
application to the jack-up reinstallation problem. A single reduction factor, RF, was applied to Hossain and Randolph’s (2009) bearing capacity factor to predict $N_{RF}$:

$$N_{RF} = RF \times N_c$$

(5.2)

This $N_{RF}$ is then applied for the determination of vertical force. For ease of application, the relatively complex RF versus z relations were simplified with the following assumptions.

1) RF remains constant from the touchdown level until the footing comes into full contact with the soil ($z=z_{contact}$), with a value defined as $RF_i$;

2) RF increases linearly between $z_{contact}$ and the footprint toe level ($z_F$);

3) RF equals unity for penetrations below $z_F$. That is, the footing resumes its full vertical capacity as the effect of the footprint ceases.

The vertical prediction method and the variation of RF with depth are expressed numerically in Equation (5.3). The concept behind this prediction method is summarised in Figure 5.7.

$$V = (RF \times N_c s_u + \gamma'z)A + \gamma'Vol,$$

(5.3)

where for $z<z_{contact}$, $RF=RF_i$;

$$z_{contact} < z < z_F, \quad RF = \frac{z-z_{contact}}{z_F-z_{contact}}(1-RF_i);$$

$$z>z_F, \quad RF=1$$

Figure 5.7: Concept of Application of a Reduction Factor to Predict the Vertical Capacity
5.4.4 Parameters Requiring Definition

In practice, $z_F$ could be obtained from a seabed scan. Alternatively, the prediction could be made based on published data. The published data on footprint geometries are mainly collected from centrifuge testing, and a summary of these records are presented in Table 3.5. The prediction of $z_{contact}$ is relatively complex, as it relates to soil properties and geometries of footings and footprints. An empirical method for the prediction of $z_{contact}$ that was developed based on the centrifuge testing on full-footing and half-footing reinstallation tests in this research is presented in Appendix C.

It was difficult to determine the $R_{Fi}$ at ground surface from the experimental results because the measured vertical forces were very sensitive at the touchdown level. At the touchdown level, only a relatively small portion of the footing was in contact with the soil, and the soil near the touchdown level was relatively soft. Therefore, $R_{Fi}$ was determined from three-dimensional small strain finite element analyses (3D-FE) of a surface footing near a footprint cavity. Only relatively small displacements are required to mobilise the bearing capacity of a surface footing. Therefore, numerical modelling using small strain finite element analyses were performed to obtain the vertical capacity of a footing at the touchdown level. The detailed development and validation of the finite element analyses are presented in Appendix D and E.

Table 5.1 summarises $N_{RF}$ and $R_{Fi}$ from the 3D-FE analyses of footings near an idealised footprint cavity in non-homogenous soils. The changes of $R_{Fi}$ with $\beta$ and footprint types are shown in Figure 5.8. $R_{Fi}$ varied with $\beta$ but was relatively insensitive to the change in footprint type.

Table 5.1: Reduced Bearing Capacity Factors and Reduction Factors from 3D-FE Analyses of Circular Footings Near a Slope in Non-Homogeneous Soil ($kD/\sum=5$)

<table>
<thead>
<tr>
<th>$\beta$</th>
<th>$N_c$ (FS)</th>
<th>$N_{RF}$</th>
<th>$R_{Fi}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>TA</td>
<td>TB</td>
</tr>
<tr>
<td>0.5D</td>
<td>7.85</td>
<td>0.43</td>
<td>0.40</td>
</tr>
<tr>
<td>1.0D</td>
<td>7.94</td>
<td>3.83</td>
<td>3.64</td>
</tr>
<tr>
<td>1.5D</td>
<td>7.93</td>
<td>7.83</td>
<td>7.72</td>
</tr>
</tbody>
</table>
Figure 5.8: Reduction Factor from 3D-FE Analyses of Circular Footings near a Slope in Non-Homogeneous Soil (kD/sum=5)

In the absence of 3D-FE analyses, RF could be estimated from Figure 5.9 based on a modified factor of safety $F^*$. The definition of $F^*$ is presented in Equation (5.4).

$$F^* = \frac{x_{\text{contact}}}{z_f F}$$

(5.4)

The $F$ values of slopes in non-homogeneous soil with a finite slope height ($z_f$) was defined according to Kuppula (1984):

$$F = \frac{S_{\text{sum}}}{\gamma z_f}$$

(5.5)

$N$ is a stability number that could be determined from Kuppula (1984) for different dimensionless strength parameters $\kappa$. However, Kuppula’s (1984) $F$ only considered the slope angle, $\theta$, and slope height, $z_s$, but not the effect of the loading width, $x_{\text{contact}}$ (a function of $B$ and $\beta$). Therefore, $F$ was modified to $F^*$ to incorporate $x_{\text{contact}}$ and $z_s$ in the equation.
Figure 5.9: Reduction Factor for Reinstallation of Strip Footing Near a Plane-Strain Footprint with Different Modified Factors of Safety

Figure 5.9 shows that there was a linear relation between $RF_i$ and the combined footprint geometric parameters. The majority of data were obtained from full-footing reinstallation tests in soil with dimensionless strength parameters of $kD/s_{um}=5.04$. An additional case, B-TB-10D, considering a different dimensionless strength parameter ($kD/s_{um}=1.96$), is also presented in Figure 5.9. The result fits into the linear trend indicating that the relationship was also valid for soil with different strengths.

5.4.5 Retrospective Simulation of Centrifuge Experiments

The prediction was compared with the vertical forces from the full-footing reinstallation tests. The comparisons are shown in Figure 5.10 to Figure 5.12. Although the method generally over-predicted the vertical force at the touchdown level, the predicted trends of an increase in the vertical force with depth were in good agreement with the test data. The prediction method gave a good estimation of vertical force throughout reinstallation, especially for $\beta \geq 1.0$. The method resulted in the least accurate prediction for $\beta=0.5D$. This could be related to the rapid change in the relatively small contact area and the lower strength of the remoulded soil filling up the void between the footing and the ground profile. This simple prediction method could also capture the change in the vertical force with footprint type. It is with use of these predicted vertical forces, that the moment and horizontal forces during reinstallation will now be formulated.
Chapter 5: Prediction of Jack-Up Reinstallation Response

Figure 5.10: Comparison of the Measured and Predicted Reduced Bearing Capacity Factors and Vertical Forces for Reinstallation near Footprint TA

Figure 5.11: Comparison of the Measured and Predicted Reduced Bearing Capacity Factors and Vertical Forces for Reinstallation near Footprint TB
5.5 PREDICTION OF MOMENT

5.5.1 Introduction to Moment Predictive Method

As indicated from the half-footing reinstallation test, footings are subjected to eccentric vertical load during reinstallation near a footprint. The moment induced from the eccentric vertical force was defined as:

\[ M = V_e \]  \hspace{2cm} (5.6)

where the eccentricity, \( e \), was measured from the centreline of the footing. The vertical force is the resultant of the soil pressure and at touchdown level, the soil pressure is acting on one side of the footing only. The eccentricity is therefore largest near the touchdown level. As penetration continued, the soil underneath the footing moved and filled in the void between the footing and the sloping ground. The magnitude of the vertical load and \( e \) were therefore affected by the soil heterogeneity. The effects of footprint geometry and soil heterogeneity on the moment responses were complex to assess. Therefore, an empirical method was developed based on the full-footing reinstallation test results.

The results from full-footing reinstallation tests and half-footing reinstallation tests suggested that the variation of normalised eccentricity, \( e/D \), with reinstallation depth were highly nonlinear. However, the \( e/D \) versus \( z \) profiles were simplified, as shown in Figure 5.13, for the prediction of the moment load throughout reinstallation.
Figure 5.13: Assumed Normalised Eccentricity versus the Depth Profile

The prediction method was developed with the following assumptions:

1) After the footing touchdown on the soil, the eccentricity increases linearly with depth until reaching \((e/D)_{\text{max}}\) at \(z_{e\text{Max}}\). \(z_{e\text{Max}}\) is the depth required to mobilise the soil strength resulting from a rapid increase in the vertical load near the touchdown level. The footprint should remain relatively intact at this depth. The reinstalation response should therefore be dominated by the footprint geometry and \(\beta\). Though some variation is observed \(z_{e\text{Max}}\) is assumed to be half of \(z_{\text{contact}}\) for simplicity. \(z_{\text{contact}}\) can be determined according to Appendix C.

2) After reaching \((e/D)_{\text{max}}\) at \(z_{e\text{Max}}\), \((e/D)_{\text{max}}\) then decreases with depth in a non-linear manner. At this stage, the geometric effect starts to reduce but remoulded soil underneath the footing contributed to the eccentricity of the loading.

3) The effect of the footprint ceases when the footing penetrates below the footprint toe level \(z_F\) and \((e/D)_{\text{max}}\) reaches zero.

The \(e/D\) versus \(z\) relation is described using the following equations:

\[
\frac{e}{D} = F_{bf} \left( \frac{e}{D} \right)_{\text{max}}
\]

(5.7)

Where for \(z \leq z_{e\text{Max}}\),

\[
F_{bf} = \frac{z}{z_{e\text{Max}}}, \quad z_{e\text{Max}} = \frac{z_{\text{contact}}}{2}.
\]

\[
z > z_{e\text{Max}}, \quad F_{bf} = \left[ 1 - \min \left( 1, \frac{z - z_{e\text{Max}}}{z_F - z_{e\text{Max}}} \right) \right]^{a_M}
\]

with \(a_M = 2\)
5.5.2 Parameters Requiring Definition

Factor $a_M$ describes the non-linearity of the decrease in $e/D$ and the concept is illustrated in Figure 5.14. By determining the best-fitting curve for the test data, $a_M=2$ is recommended. For simplicity, $a_M$ was also assumed to be independent of footprint type and $\beta$.

![Figure 5.14: The Effect of $a_M$ on the Change of Eccentricity with Depth](image)

The maximum normalised eccentricity, $(e/D)_{\text{max}}$, can be obtained from Figure 5.15. It was established from the full-footing reinstallation tests and 3D-FE analyses (details of which can be found in Appendix E). A general trend of $(e/D)_{\text{max}}$ increasing as $\beta$ decreased was observed. However, the $(e/D)_{\text{max}}$ values collected from the two studies were relatively scattered. Whilst $(e/D)_{\text{max}}$ determined from 3D-FE was independent of footprint type, $(e/D)_{\text{max}}$ determined from full-footing reinstallation tests varied with footprint type and offset distance $\beta$. This occurred because the 3D-FE output was based on small strain analyses of footings at the touchdown level. As a result, $(e/D)_{\text{max}}$ only reflected the geometric effect at the touchdown level.

For $\beta=0.5$, the $(e/D)_{\text{max}}$ was relatively sensitive to the change of footprint type, so $(e/D)_{\text{max}}$ was therefore conservatively assumed to equal 0.5. In theory, for a footing at $\beta=0.5$, $(e/D)_{\text{max}}$ should occur at the touchdown level and should always be 0.5. However, at this offset distance, only a very small area of the footing was in contact with the soil, and the vertical load obtained from testing was relatively sensitive to the reinstallation depth. On the other hand, the $(e/D)_{\text{max}}$ was relatively insensitive to the change of footprint type for the case of $\beta=1.5D$. The moment was only contributed by the eccentricity from the slightly skewed bearing resistance along the footing base. The
(e/D)\textsubscript{max} was therefore relatively small, and a minimum (e/D)\textsubscript{max} of 0.05 is conservatively recommended. For reinstallation at an offset between β=0.5D and 1.5D, (e/D)\textsubscript{max} could be obtained by interpolation from the prediction curve (Figure 5.15).

![Figure 5.15: Maximum Normalised Eccentricity from a Full-Footing Reinstallation Test](image)

**5.5.3 Retrospective Simulation of Centrifuge Experiments**

This moment prediction method was applied together with the vertical force prediction method presented in Section 5.4.3. A comparison was made with the full-footing reinstallation test results presented in Chapter 3 to review its performance for a wide range of problems with different footprint geometries and offset distances. The comparison is shown in Figure 5.16 to Figure 5.18.

The prediction provided a good approximation to the trend of the change in moment with depth. The method predicted the moment during relatively shallow depth reinstallation with good accuracy, which involved an increase in the moment from zero to its maximum value (M\textsubscript{max}) at \( z_{e\text{Max}} \). The method generally over-estimated the moment for reinstallation below \( z_{e\text{Max}} \). This was likely related to conservative assumptions made about (e/D)\textsubscript{max} and a\textsubscript{M}. However, the agreement improves with increasing footprint size from type TA to TC.
Figure 5.16: Comparison of the Measured and Predicted Eccentricities and Moments for Reinstallation near Footprint TA

Figure 5.17: Comparison of the Measured and Predicted Eccentricities and Moments for Reinstallation near Footprint TB
The maximum moment, \( M_{\text{max}} \), obtained from the predictions are compared with the test results in Table 5.2. A comparison on the depth where maximum moment occurred, \( z_{M_{\text{MMax}}} \), are also presented in Table 5.2. The maximum difference between the predicted and measured \( M_{\text{max}} \) was within 25% on the conservative side. For the critical offset distance, the difference between the predicted and measured \( z_{M_{\text{MMax}}} \) was within 20%. Therefore, this simple prediction method is useful in predicting the moment during footing reinstallation.

**Table 5.2: Comparison on \( M_{\text{max}} \) from Prediction and Testing**

<table>
<thead>
<tr>
<th>Test</th>
<th>Predicted ( M_{\text{max}} ) (MNm)</th>
<th>Measured ( M_{\text{max}} ) (MNm)</th>
<th>Difference in ( M_{\text{max}} ) (%)</th>
<th>Predicted ( z_{M_{\text{MMax}}}/D ) (m)</th>
<th>Measure ( z_{M_{\text{MMax}}}/D ) (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TA-10D</td>
<td>10.46</td>
<td>12.52</td>
<td>+16.4%</td>
<td>0.03</td>
<td>0.03</td>
</tr>
<tr>
<td>TA-15D</td>
<td>7.55</td>
<td>5.66</td>
<td>+25.0%</td>
<td>0.01</td>
<td>0.03</td>
</tr>
<tr>
<td>TB-05D</td>
<td>14.79</td>
<td>15.90</td>
<td>-7.6%</td>
<td>0.20</td>
<td>0.17</td>
</tr>
<tr>
<td>TB-10D</td>
<td>14.24</td>
<td>11.91</td>
<td>+16.4%</td>
<td>0.04</td>
<td>0.05</td>
</tr>
<tr>
<td>TB-15D</td>
<td>7.59</td>
<td>8.53</td>
<td>-12.3%</td>
<td>0.01</td>
<td>0.01</td>
</tr>
<tr>
<td>TC-05D</td>
<td>16.60</td>
<td>17.32</td>
<td>+1.2%</td>
<td>0.35</td>
<td>0.40</td>
</tr>
<tr>
<td>TC-10D</td>
<td>25.60</td>
<td>25.31</td>
<td>-4.4%</td>
<td>0.20</td>
<td>0.20</td>
</tr>
</tbody>
</table>
5.6 PREDICTION OF HORIZONTAL FORCE

5.6.1 Introduction to Horizontal Force Predictive Method

Based on the observation from full-footing reinstallation tests, the horizontal responses were influenced by the size of the horizontally moving soil block and the unbalanced lateral earth pressure. The change of horizontal force with reinstallation depth was highly non-linear because of the complex soil movement. Therefore, it is proposed to predict the horizontal force from a simplified relationship between inclination angle and penetration depth. The horizontal force is related to the vertical force in terms of the inclination angle, \( \alpha \):

\[
H = V \tan \alpha
\]  

(5.8)

The \( \alpha \) versus \( z \) profile was simplified, as shown in Figure 5.19, for the prediction of horizontal force.

![Figure 5.19: Assumed Inclination Angle versus Depth Profile](image)

The following assumptions were made to facilitate the formulation of the horizontal force prediction method:

1) As the footing touches down onto the soil, \( \alpha \) increases rapidly with the soil movement from the failed slope and \( \alpha \) reaches its maximum value at \( Z_{\text{contact}} \).

2) After reaching \( \alpha_{\max} \), the soil movement from slope failure reduces but the effect from unbalanced earth pressure increases. Therefore, \( \alpha \) is assumed to decrease gradually with depth.
3) The effect of the footprint ceased at a depth two times the footprint toe level $z_F$; therefore, $\alpha$ decreases to zero at $2z_F$.

The following equations were developed to determine $\alpha$ over $z$.

$$\alpha = F_b \alpha_{\text{max}}$$

(5.9)

where for $z \leq z_{\text{contact}}$, $F_b = \left( \frac{z}{z_{\text{contact}}} \right)^{0.5}$,

for $z > z_{\text{contact}}$, $F_b = \left[ 1 - \left( \frac{z - z_{\text{contact}}}{2z_F - z_{\text{contact}}} \right) \right]^{a_H}$

5.6.2 Parameters Requiring Definition

Whilst the rate of increase in $\alpha$ was assumed to be the same for all cases, the reduction of $\alpha$ with depth varied with offset distance and footprint type. The effect of change in $a_H$ to the $\alpha$ profile is shown in Figure 5.20. $a_H$ was determined from best-fitting the data collected from a full-footing reinstallation test. The suggested values are presented in Figure 5.21. $a_H$ was sensitive to footprint type but independent of $\beta$.

![Figure 5.20: The Effect of $a_H$ on the Change in the Inclination Angle with Depth](image)
The maximum inclination angle, $\alpha_{\text{max}}$, could be obtained from Figure 5.22. Because $\alpha_{\text{max}}$ was more sensitive to the change of footprint type than $\beta$, only one $\alpha_{\text{max}}$ versus slope angle prediction line was proposed for all values of $\beta$.

5.6.3 Retrospective Simulation of Centrifuge Experiments

The horizontal force and vertical force prediction method were applied together and compared with the full-footing reinstallation test results. The comparisons are presented
in Figure 5.23 to Figure 5.25. The predicted trends of the change in the horizontal force with depth were in agreement with the test results. The method generally overestimated the horizontal force for $\beta=1.5$, due to the simplification made to the prediction of $\alpha_{\text{max}}$. For the cases of footprint TC, which involved the steepest and deepest footprint, the predicted values were larger than the measured values, and $H_{\text{max}}$ also occurred at a shallower depth. This was because the change in the horizontal force was related to the complex change in the failure mechanisms (changing from overall slope failure to skewed bearing failure). The method still provided a conservative approximation to such a complicated problem.

![Figure 5.23: Comparison of the Measured and Predicted Inclinations and Horizontal Forces for Reinstallation near Footprint TA](image-url)
The predicted and measured $H_{\text{max}}$ values are compared in Table 5.3. This simple prediction method generally over-predicted the $H_{\text{max}}$, but the maximum over-prediction was within 37%. This over-prediction occurred for reinstallation near a relatively steep footprint (TC in this comparison) or at a relatively large offset distance ($\beta \geq 1.5D$). The predicted depth of occurrence of maximum moment occurred, $z_{H\text{Max}}$, were generally deeper than the measured $z_{H\text{Max}}$. This method could be further improved by conducting a more comprehensive study on the change of $\alpha_{\text{max}}$ with slope angle and offset distance.
\( a_H \) could also be calibrated with a larger amount of data to more accurately describe the reduction in \( \alpha \) with reinstallation depth. With a more accurate prediction of \( \alpha_{\text{max}} \) and \( a_H \), the agreement between the predicted and measured values could be improved. However, this simple prediction method could still be used as a first estimate, so further improvement of the method is not performed here.

Table 5.3: Comparison of \( M_{\text{max}} \) Values Obtained from Prediction and Testing

<table>
<thead>
<tr>
<th>Test</th>
<th>Predicted ( H_{\text{max}} ) (MN)</th>
<th>Measured ( H_{\text{max}} ) (MN)</th>
<th>Difference in ( M_{\text{max}} ) (%)</th>
<th>Predicted ( z_{H_{\text{max}}}/D ) (m)</th>
<th>Measure ( z_{H_{\text{max}}}/D ) (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TA-10D</td>
<td>0.45</td>
<td>0.46</td>
<td>-0.7%</td>
<td>0.27</td>
<td>0.17</td>
</tr>
<tr>
<td>TA-15D</td>
<td>0.43</td>
<td>0.40</td>
<td>+8.3%</td>
<td>0.27</td>
<td>0.16</td>
</tr>
<tr>
<td>TB-05D</td>
<td>1.08</td>
<td>0.74</td>
<td>+31.9%</td>
<td>0.34</td>
<td>0.25</td>
</tr>
<tr>
<td>TB-10D</td>
<td>0.88</td>
<td>0.88</td>
<td>0.05%</td>
<td>0.34</td>
<td>0.31</td>
</tr>
<tr>
<td>TB-15D</td>
<td>0.80</td>
<td>0.60</td>
<td>+25.8%</td>
<td>0.34</td>
<td>0.27</td>
</tr>
<tr>
<td>TC-05D</td>
<td>1.92</td>
<td>1.73</td>
<td>+10.1%</td>
<td>0.31</td>
<td>0.40</td>
</tr>
<tr>
<td>TC-10D</td>
<td>2.30</td>
<td>1.46</td>
<td>36.4%</td>
<td>0.14</td>
<td>0.20</td>
</tr>
</tbody>
</table>

#### 5.7 EXAMPLE APPLICATION OF THE PREDICTION METHOD

The ability of the prediction method to be used for real footprint scenarios, that is, to a footprint with remoulded soil strength, will be discussed in this section by retrospectively simulating two other experimental studies. These studies were not used in the development of the model and therefore provide further model verification.

##### 5.7.1 Retrospective Simulation of the Tests of Cassidy et al. (2009)

The first one considered was Cassidy et al. (2009), which involved reinstallation of a conical base footing near a real footprint formed in slightly over-consolidated soil. The dimensionless strength parameter of the soil adopted in the test was \( kD/s_{\text{um}}=5.03 \), and the details of the test are presented in Chapter 2. The parameters adopted in the prediction are presented in Table 5.4.
Table 5.4: Parameters Adopted in the Retrospective Simulation of the Test of Cassidy (2009)

<table>
<thead>
<tr>
<th>Test</th>
<th>OTDR2</th>
<th>OTDR7</th>
<th>OTDR12</th>
</tr>
</thead>
<tbody>
<tr>
<td>$s_u$</td>
<td>$s_u=6.5+1.8z$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$D$</td>
<td>18.18m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$x_i/D$</td>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$z_F/D$</td>
<td>0.33</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Footprint geometry</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$z_{contact}/D$</td>
<td>0.15</td>
<td>0.07</td>
<td>5E-4</td>
</tr>
<tr>
<td>$\beta$</td>
<td>0.5D</td>
<td>1.0D</td>
<td>1.5D</td>
</tr>
<tr>
<td>$R_{F1}$</td>
<td>0.05</td>
<td>0.46</td>
<td>0.97</td>
</tr>
<tr>
<td>$(e/D)_{max}$</td>
<td>0.50</td>
<td>0.15</td>
<td>0.05</td>
</tr>
<tr>
<td>$a_M$</td>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\alpha_{max}$</td>
<td>0.05 rad</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$a_H$</td>
<td>0.30</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The $s_u$ profile presented in Cassidy (2009) was $s_u=7.5+2.0z$ and it was considered to be slightly higher than t-bar characterisation test data and represented the upper bound soil strength. In this study, a detailed interpretation was performed on the original test data and a revised $s_u$ profile of $s_u=6.5+1.8z$ was adopted.

Figure 5.26 to Figure 5.28 show the comparison of the predicted and measured loadings from Cassidy et al. (2009). Although the vertical force induced by reinstallation near a real footprint was expected to be lower (remoulded soil with relatively low strength), the prediction method did not result in a significant over-estimation. The prediction was in excellent agreement with Cassidy et al. (2009), except for the cases of $\beta=0.5D$ in which under-estimation of the vertical force at a relatively shallow reinstallation depth ($z<0.22D$) was observed. It is likely to be related to the fundamental difference in the footing geometry. The method was developed based on testing data from reinstallation of a flat base footing, while a spudcan footing with conical base was adopted in Cassidy et al. (2009). However, the overall trends for the change in vertical force with depth from the prediction method and the experimental study were in good agreement.

The lower vertical force from reinstallation near the real footprint resulted in a smaller moment. The prediction method, which only considered the effect of footprint geometry, therefore provided a relatively conservative approximation to the moment. However, the difference in $M_{max}$ was within 35%. The performance of the horizontal force prediction method was similar in both the real footprint and idealised footprint cases. The prediction method generally over-predicted the horizontal force for the case of $\beta=1.5D$ and under-estimated it for $\beta=0.5D$. Again, it is because the effect of soil heterogeneity was ignored in the development of $\alpha_{max}$ and $a_H$. 

---

\(a\) The $s_u$ profile presented in Cassidy (2009) was $s_u=7.5+2.0z$ and it was considered to be slightly higher than t-bar characterisation test data and represented the upper bound soil strength. In this study, a detailed interpretation was performed on the original test data and a revised $s_u$ profile of $s_u=6.5+1.8z$ was adopted.
Chapter 5: Prediction of Jack-Up Reinstallation Response

Figure 5.26: Comparison of the Measured (Cassidy et al. 2009) and Predicted Reduced Bearing Capacity Factors and Vertical Forces for Reinstallation Near a Real Footprint

Figure 5.27: Comparison of the Measured (Cassidy et al. 2009) and Predicted Eccentricities and Moments for Reinstallation Near a Real Footprint
Chapter 5: Prediction of Jack-Up Reinstallation Response

5.7.2 Retrospective Simulation of Real Footprint Test of this Thesis

The other experimental study involved the reinstallation of a flat-base footing near a real footprint formed in slightly over-consolidated soil (kD/s_{um}=1.95). The testing details will be presented in Chapter 8. The parameters adopted in the prediction are presented in Table 5.5.

The comparisons between the prediction and the reinstallation test results are presented in Figure 5.29 to Figure 5.31. An offset distance of 1.0D was adopted in the tests. Again, only a slight over-estimation was observed in the predicted vertical force because the effect of soil heterogeneity was not considered in the prediction method. The moment from the reinstallation test remained almost constant below z_f rather than decreasing to zero as in the prediction method and it is due to the effect of soil heterogeneity. This analytical prediction method does not extend to this type of behaviour. However, the predicted M_{max} was in excellent agreement, and the maximum difference was within 5%. Although a slight over-estimation of the horizontal force was observed, the overall trends for the changes in the horizontal force for the predictions and the test results were in excellent agreement.
Table 5.5: Parameters Adopted in the Retrospective Simulation of the Real Footing Reinstallation Test (Details Presented in Chapter 9)

<table>
<thead>
<tr>
<th>Test</th>
<th>B-R-1</th>
<th>B-R-2</th>
</tr>
</thead>
<tbody>
<tr>
<td>$s_u$</td>
<td>$s_u=2.00+0.65z$</td>
<td>$s_u=1.20+0.40z$</td>
</tr>
<tr>
<td>$D$</td>
<td>6m</td>
<td></td>
</tr>
<tr>
<td>$x_f/D$</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>$z_f/D$</td>
<td>0.33</td>
<td></td>
</tr>
<tr>
<td>Footprint geometry</td>
<td>TB</td>
<td></td>
</tr>
<tr>
<td>$z_{contact}/D$</td>
<td>0.07</td>
<td></td>
</tr>
<tr>
<td>$\beta$</td>
<td>1.0D</td>
<td></td>
</tr>
<tr>
<td>$R_{f_1}$</td>
<td>0.46</td>
<td></td>
</tr>
<tr>
<td>$(e/D)_{max}$</td>
<td>0.15</td>
<td></td>
</tr>
<tr>
<td>$a_M$</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>$\alpha_{max}$</td>
<td>0.05 rad</td>
<td></td>
</tr>
<tr>
<td>$a_H$</td>
<td>0.30</td>
<td></td>
</tr>
</tbody>
</table>

Figure 5.29: Comparison of the Measured (Chapter 9) and Predicted Reduced Bearing Capacity Factors and Vertical Forces for Reinstallation near a Real Footprint
5.8 SUMMARY OF THE PREDICTION METHOD PROCEDURES

Based on the design charts and equations presented above, a design procedure for the prediction of vertical, horizontal and moment load is proposed, consisting of the following steps:
1) identify the soil strength of the undisturbed soil;

2) determine the key footprint geometric parameters (footprint diameter $x_F$ and footprint depth $z_F$) from a seabed scan or approximate using Table 3.5;

3) determine the penetration depth when the footing comes into full contact with the soil ($z_{contact}$), according to Appendix C;

4) for a design offset distance, $\beta$, determine the reduction factor, $R_{F_i}$, from Figure 5.9 or Table 5.1;

5) assess the vertical force using Equation (5.3);

6) assess the maximum normalised eccentricity $(e/D)_{max}$ from Figure 5.15;

7) assess the moment using Equations (5.6) and (5.7);

8) assess $a_H$ from Figure 5.21 and $\alpha_{max}$ from Figure 5.22;

9) assess the horizontal force using Equations (5.8) and (5.9).

5.9 CONCLUDING REMARKS

This chapter presents a simple method for the prediction of vertical and horizontal forces and the moment during jack-up reinstallation. The jack-up reinstallation problem was simplified as a flat-base footing fully fixed from rotation and sliding by reinstallation near conical-shaped footprints. The method was applicable for the prediction of shallow depth reinstallation response, that is, reinstallation depths shallower than the depth of two times footprint toe level. The shallow depth reinstallation response was governed by the footprint geometry, rather than the soil heterogeneity. The prediction method was developed from the full-footing reinstallation test presented in Chapters 3 and 4. There are only two key input parameters for the method: the strength of the undisturbed soil and the geometry of the footprint. A retrospective comparison of the prediction model with two other experimental studies not used in its determination confirmed that the method performed satisfactorily for the prediction of shallow depth reinstallation response near real footprints.

However, the assumptions made to simplify the problem impose limitations on the application of the prediction methods. These limitations are discussed here, along with suggestions for ways to improve the prediction method:

- By assuming the footing was fully-fixed, the decrease in the loading as a function of jack-up movement was ignored. The predicted reinstallation response was therefore conservative. Later in Chapter 9 of this thesis the
prediction will be adjusted to allow for the change of jack-up reinstallation response with jack-up structural stiffness.

- The assumption of ignoring the effect of soil heterogeneity during shallow depth reinstallation was considered to be acceptable because the soil strength around the footprint could be the same or higher than the undisturbed soil. Gan (2010) demonstrated that if the elapsed time was relatively long (100 years), the soil around the footprint regained its strength, and the final strength could be higher than the undisturbed soil. However, for reinstallation taking place after a relatively short elapsed time (1 year in Gan 2010), ignoring the effect of soil heterogeneity leads to potential over-estimation of the vertical, horizontal and moment loads. However, the potential over-estimation provided the necessary safety margin for the complex problem.

- The prediction method could be enhanced by incorporating differences due to soil heterogeneity, specifically by adding another reduction factor $RF_s$ to Equation (5.3). Such a factor could be larger than or smaller than unity depending on the stress history of the undisturbed soil and the operation procedures of the previous jack-up unit, with contour plots form the testing of Gan (2009) useful in determining this.

- The moment fluctuated around its maximum value for penetration below the footprint toe level, rather than decreasing to zero as assumed. $z_F$ in Equation (5.7) could be adjusted to a greater depth. The adjusted depth should be related to the maximum penetration depth of the initial jack-up installation ($z_{FC}$).

- The prediction method was calibrated from 7 experimental tests. The accuracy of the method could be improved by calibrating the key variables ($RF_i$, $(e/D)_{max}$, $\alpha_{max}$, $a_M$ and $a_H$ with more experimental results and/or large deformation finite element analyses. Calibration to different soil conditions may be necessary.

- For deeper reinstallation (near $z_{FC}$), the reinstallation response could be influenced by the firm layer formed near $z_{FC}$. The footing could experience another peak horizontal force and a moment in the reverse direction. The prediction method is not applicable for the prediction of the deep reinstallation response. In such cases, the response could be determined from physical modelling in a centrifuge or large-deformation finite element analyses.
CHAPTER 6. DEVELOPMENT OF A REAL-TIME HYBRID TESTING METHOD

6.1 INTRODUCTION

A primary objective of this study is to understand the influence of the structural stiffness of a jack-up unit during its reinstallation near an existing footprint. The structural stiffness is a function of the structural properties of the jack-up unit, the jack-up configuration (e.g. height of hull) and the foundation fixity. This study aimed to investigate the response of a jack-up unit during typical reinstallation procedures. The scenario studied was two jack-up legs located further away from the footprint are first installed to gain some foundation fixity, followed by the installation of the third jack-up leg near the existing footprint (Figure 6.1).

![Figure 6.1: One Jack-Up Leg Reinstalled nearby Existing Footprint (Contour of Footprints after Gan 2009)](image)

Experimentally, the structural stiffness of the jack-up unit could be considered by adopting a full 3-legged jack-up model in centrifuge tests, a method used in measuring combined loading of a jack-up by Dean et al. (1996) and Bienen et al. (2006). However, application to study reinstallation near a footprint is not straightforward.

The first challenge is modelling the complex jack-up leg hull connection. The jack-up leg needs to be installed, therefore controlled vertical movement through the hull is required. The connection should also allow for horizontal and rotational interaction with the rest of the jack-up unit. In offshore conditions, the jack-up leg is connected to the hull through a rack-and-pinion connection (Figure 6.2). However, it is difficult to
fabricate such a complicated connection in a reduced scale jack-up model. All of the published test results on full jack-up models considered rigid jack-up legs-hull connections.

The second challenge arises from the number of actuator and model connections required within a test. Modelling the complete jack-up reinstallation procedures requires at least three actuator and model connections. The first connection is for the actuator to install either a single jack-up leg or the full 3 legged jack-up model to create a footprint. The second connection would be for the installation of the two legs of the jack-up model not close to the footprint. Finally, the actuator needs to be connected to the one leg for installation near the existing footprint. This is extremely complex. For a standard beam centrifuge facilities with only one actuator-tool connection, the centrifuge must be ramped down twice for connection changes within one test. This could potentially alter the stress state of the soil sample and of particular concern is the soil strength under the footprint.

In addition to these technical challenges, testing a full 3-leg jack-up model is inefficient for conducting a parametric study. Relatively large testing space is required and also a large number of 3-leg jack-up models need to be built for a parametric study on a range of structural stiffness.
A hybrid approach is possible, with a single spudcan and leg tested experimentally and a structural stiffness prescribed at the actuator connection (Figure 3.2). The simplest form is to perform this mechanically. Examples include use of a hinge connection allowing for rotational movement of the jack-up leg for combined loading tests by Cassidy and Byrne (2001), Cassidy et al. (2004) and Govoni et al. (2010), or the free-to-slide roller connection adopted for reinstallation tests by Gaudin et al. (2008). The hinged connection modelled a jack-up leg with no rotational stiffness and the roller connection modelled no horizontal stiffness.

However, a mechanical connection is simple (relatively) to design for one degree-of-freedom, but complex for the combined vertical, horizontal and moment required in the experiments proposed in this research (Figure 3.2). Furthermore, to change the stiffness for different jack-up configurations, installations or any non-linear elastoplastic response is not feasible mechanically.

A real-time hybrid testing method was therefore developed to efficiently conduct a parametric study of a wide range of realistic jack-up structural stiffness. The critical element of the spudcan and leg being installed near the footprint was modelled physically, whilst the remainder of the jack-up unit was modelled numerically.

In this chapter the underlying principles and the design criteria of this novel real-time hybrid testing is discussed. A new vertical-horizontal-rotation actuator system (named the VHM actuator) was required and details of its design, verification and application are provided. Furthermore, control of the tests was found to be critical and the real-time control system developed is presented in detail in this chapter. Results of experimental
validation of the system are provided to illustrate the effectiveness of the real-time hybrid testing methodology.

6.2 REAL-TIME HYBRID TESTING

6.2.1 A Review on Real-Time Hybrid Testing

Given the large size of structure and the large magnitude of loading it is often impractical to physically model the response of a full civil engineering structure. This is definitely the case for jack-up units. Hybrid physical-numerical modelling is a solution that combines a physical test on the critical element (which is usually extremely non-linear) with a numerical model on the remainder of the structure (Figure 6.4). During a test the numerical calculation and the physical tests are run together in either an expanded time scale or they can be made to interact in real-time. The former is referred to as pseudo-dynamic hybrid testing. In the latter, the loading and response in the test is occurring at the same rate as in the prototype. In the literature this technique has been referred to as real-time hybrid testing, real-time hybrid simulation and even real-time dynamic substructuring (Saouma and Sivaselvan 2008). In this chapter, this technique will be simply referred to as real-time hybrid testing.

Figure 6.4: Example Application of Hybrid Testing to a Knee-Braced Frame (after Blakeborough et al. 2001)
Early development of hybrid testing was for the modelling of the seismic response of a structural system. The prime objective was to realistically model the response of full scale or large structural systems with limited actuator power. The first hybrid testing was performed by Hakuno et al. (1969). An electromagnetic actuator was used to load the physical model and an analogue computer was used to analyse the numerical model. With the introduction of digital computers in the 1970s, the hybrid testing method was expanded and validated by researchers, mainly in Japan and the United States. Early development used actuators of relatively low power, therefore loading was applied to the structure in expanded time scales as pseudo-dynamic hybrid testing. Detailed discussion can be found in Takanashi (1975), Mahin and Shing (1985) and Takanashi (1985).

In pseudo-dynamic hybrid testing, the requirement of a relatively low power actuator allowed for large scaled model test to be performed using conventional testing device in a moderately sized laboratory. Furthermore, detailed observation of the structural behaviour and failure could be made throughout the test. However, the method is not suitable for accurate modelling of rate dependent systems. Therefore, real-time hybrid testing, which was extended from pseudo-dynamic testing, was developed in the 1990s for modelling rate dependent systems. While pseudo-dynamic hybrid testing is displacement controlled, real-time hybrid testing is displacement and velocity controlled, with the physical testing and the numerical modelling using a common time axis. Real-time hybrid testing was first achieved by Nakashima et al. (1992). Successful implementation with a dynamic load was reported by Horiuchi et al. (1999).

In a real-time hybrid test, actuators apply displacement to a physical model, and the physical test interacts with a numerical model in real-time by means of a feedback control loop. In this way the complete structural system is tested. Figure 6.5 shows the control loop in its basic form. The loop starts by applying external load (e.g. earthquake load) to the numerical model. The response at the common point between the physical model and numerical model is evaluated using a time stepping scheme (Figure 6.6). The displacements at the common point are imposed upon the physical model by the actuator in real-time (i.e. over the same period of time that the numerical model is stepping). The restoring forces are measured directly from the physical model at the end of the time-step. These forces are feedback and imposed to the common point of the numerical model. The response of the numerical model to external load and feedback forces is evaluated for the end of the next time step. Looping continues until the external load and the response cease.
Figure 6.5: Example Application of Real-time Hybrid Test Control Loop to a Knee-Braced Frame (after Blakeborough et al. (2001))

Figure 6.6: Basic Time Stepping Scheme

Figure 6.7 shows the typical apparatus required for a real-time hybrid testing system. Successful implementation of real-time hybrid testing require three components: (i) a fast solution of the numerical model, (ii) a high performance loading system and measuring device on the physical model and (iii) a high-speed network that links the computation machine to the load application hardware.
Real-time hybrid testing is subjected to delay like any other testing system. Some sources of delay are negligible, such as the delay in digital signal processing and data acquisition. However, the delay in the time stepping process and the finite actuator response time may have a significant influence on the testing. Figure 6.8 illustrates the concept of these two sources of delay. The control loop ideally requires the actuator to impose the demand displacements on the physical model within the duration of the numerical model time step. However, this cannot be achieved in reality. Furthermore, the actuator cannot react instantaneously as prescribed by the numerical model. This can be particularly critical when using a hydraulic actuator and the delay is typically in the order of 10-20ms (Darby et al. 2001). Horiuchi et al. (1999) demonstrated that such delay can induce negative damping. If the response time of the actuators exceeds the critical delay time of the system, that is when the negative damping become greater than the inherent structural damping, the system can become unstable and the response oscillate with exponentially increasing magnitude. Therefore, the system can only be applied to structures with a small stiffness or a large damping. Delay compensation schemes have been developed to predict and cancel the delay in the control loop in order to achieve a stable system. The methodology has been proven to be successful and examples can be found in Horiuchi et al. (1999), Darby et al. (2001) and Wallace et al. (2005). This allows the application of real-time hybrid testing to wider varieties of structural systems.
Recent advancement has been made in understanding experimental error, improving delay compensation algorithms and substructuring techniques. More complex and large scale tests have also been performed in geographically distributed laboratories through networked hybrid testing. Examples include NEE MOST in US (Kwon et al. 2005) and KOCED in Korea (Park et al. 2004). The use of real-time hybrid testing has gained popularity in pursuing accurate modelling of dynamic behaviour of complex mechanical and structural systems, such as bridges, buildings and aircrafts (Blakeborough et al. 2001, Wagg et al. 2008, Van der Auweraer et al. 2008, Ayari 2008).

### 6.2.2 Application of Hybrid Testing in Geotechnical Modeling

Table 6.1 summarises the application of hybrid testing in geotechnical modelling. The first application of hybrid testing in geotechnical engineering was in 1975 and investigated the dynamic performance of a pile foundation (Mochizuki 1975). The subsequent development concentrated on one-dimensional liquefaction potential evaluation. In such study, the liquefiable layer was physically modelled by a soil element test, while a numerical model was applied to the remaining layers. Figure 6.9 shows an example of the testing set up. The deformation of soil caused by liquefaction both during and after an earthquake was evaluated. Various researchers concluded that hybrid testing provided accurate prediction of dynamic soil response without the use of idealised constitutive relations (e.g. Takahashi et al. 2003, Kazama et al. 2004, Kwon et al. 2007).
Table 6.1: Application of Hybrid Testing in Geotechnical Modelling

<table>
<thead>
<tr>
<th>Reference</th>
<th>Study</th>
<th>Testing Methodology</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mochizuki (1975)</td>
<td>Dynamic response of pile in clay</td>
<td>Modelling a reduced scaled pile (100mm diameter and 748.5mm long)</td>
</tr>
<tr>
<td>Katada and Hakuno (1982)</td>
<td>Liquefaction study of sand</td>
<td>Cyclic triaxial test</td>
</tr>
<tr>
<td>Kusakabe and Morio (1995)</td>
<td>Liquefaction study of sand</td>
<td>6 hollow torsional shear test and numerical soil layer</td>
</tr>
<tr>
<td>Adachi et al. (1998)</td>
<td>Liquefaction study of sand</td>
<td>Torsion cyclic test and numerical soil layer</td>
</tr>
<tr>
<td>Fujii et al. (2000)</td>
<td>Liquefaction study of embankments founded on sandy deposit</td>
<td>Direct shear tests and numerical soil layer</td>
</tr>
<tr>
<td>Sento et al. (2002)</td>
<td>Liquefaction study of sand</td>
<td>Element test and numerical soil layer</td>
</tr>
<tr>
<td>Yamaguchi et al. (2002)</td>
<td>Liquefaction study of Kobe Port</td>
<td>Element test on Kobe Port sand sample and numerical soil layer</td>
</tr>
<tr>
<td>Kobayashi et al. (2002)</td>
<td>SSI of bridge pier found in sand subjected to earthquake load</td>
<td>1:3.33 scaled shake table test with soil modelled physically in laminar box</td>
</tr>
<tr>
<td>Takahashi et al. (2003)</td>
<td>Liquefaction study of multi-layered ground</td>
<td>3 simple shear test and numerical soil layer</td>
</tr>
<tr>
<td>Kazama et al. (2004)</td>
<td>Liquefaction study of multi-layered ground</td>
<td>Multi simple shear test and numerical soil layer</td>
</tr>
<tr>
<td>Spencer et al. (2006)</td>
<td>SSI of bridge structure subjected to earthquake load</td>
<td>Networked hybrid testing with soil modelled numerically</td>
</tr>
<tr>
<td>Kwon et al. (2007)</td>
<td>Consolidation study of multi-layered ground</td>
<td>Series of consolidometer 1D consolidation test and numerical soil layer</td>
</tr>
<tr>
<td>Ohtomo et al. (2008)</td>
<td>SSI of subway structure subjected to earthquake load</td>
<td>Shake table with soil modelled numerically by non-linear finite element programme</td>
</tr>
<tr>
<td>Wang et al. (2009)</td>
<td>SSI of two storey building on raft footing subjected to earthquake load</td>
<td>Shake table test with soil modelled numerically</td>
</tr>
</tbody>
</table>
On-the-other-hand, the application to the study of soil-structure-interaction (SSI) is relatively limited. Five studies were found and they all reported on dynamic SSI. Mochizuki (1975) physically modelled the soil and the pile with the rest of the building structure modelled numerically as lumped masses. Kobayashi et al. (2002) also physically modelled the pier and pile foundation in the laminar box of a shake table with the rest of the bridge structure modelled numerically (Figure 6.10). In other studies the soil and foundation system was modelled numerically by means of a finite element programme or lumped mass model programme. Figure 6.11 shows the networked hybrid testing presented by Spencer et al. (2006) for the investigation of dynamic response of bridge structure. Ohtomo et al. (2008) compared the hybrid test results with a shake table test and verified the reliability of the methodology. Wang et al. (2009) compared the hybrid test results with a full numerical model and concluded that hybrid testing produces satisfying results in SSI analysis (Figure 6.12).

Figure 6.9: Hybrid Testing on Dynamic Response of Multi-layered Ground Using Simple Shear Element Test (after Takahashi et al. 2003)
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Figure 6.10: Hybrid Testing on Soil-Structure-Interaction of Bridge Pier Found in Sand (Kobayashi et al. 2002)

Figure 6.11: Hybrid Testing on Soil-Structure-Interaction of Bridge Structure (Spencer et al. 2006)
6.3 REAL-TIME HYBRID TESTING OF A JACK-UP UNIT

This study extended the real-time hybrid testing methodology to investigate the response of a jack-up reinstallation near a footprint. The methodology was developed to allow realistic simulation of soil stress level and drainage condition in an offshore scenario. A novel real-time hybrid testing system incorporating the beam centrifuge facility at UWA has been developed. Centrifuge modelling was employed to realistically simulate the foundation response with a soil stress level comparable to prototype conditions. The system allows the measured load to be feedback into the control loop and the actuator to transfer the load onto the physical model in real-time. The rate dependent response of the model could therefore be correctly modelled. This was critical as an installation speed similar to the offshore undrained condition could be maintained.

The critical component requiring physical modelling was the interaction of the leg/spudcan and the footprint, as this behaviour cannot be numerically predicted. The remainder of the jack-up unit was modelled numerically and was described by a load-displacement relationship. Real-time hybrid testing was achieved through the exchange of loads and displacements at a common point on the physical and numerical model. This concept is illustrated for the jack-up in Figure 6.13. Firstly, the loading (vertical, horizontal and moment loads) at the common point, CP, was derived through measurements in the physical jack-up model. These were provided to the numerical model, which in turn calculated the vertical and horizontal displacements and rotations ($v$, $h$, and $\theta$) to be imposed on the physical jack-up leg during the next step. An actuator capable of applying this three degree-of-freedom actuator was therefore required (and will be discussed in Section 6.4).
Figure 6.13: Concept of Real-time Hybrid Testing in a Jack-up Reinstallation near an Existing Footprint

Figure 6.14 shows the outline of the system and the communication of forces and displacements between the hardware. The control loop is presented in Figure 6.15. It controls the flow loop outlined in Figure 6.13, and allows the hybrid tests to be run in the beam centrifuge. The control loop has the following features:

- The loading acting on the physical jack-up model were measured from strain gauges;
- The load readings were transferred to the “DigiDAQ Box” which is an analogue-to-digital converter (ADC) system;
- The load readings were then transferred to the “flight computer” mounted on the central platform of the centrifuge (flight PC) through an Ethernet connection;
- In the flight PC, the load reading was streamed to two programmes;
  1. the data logging system DigiDAQ which is a purely data logging system;
  2. the real-time control algorithm and numerical model programme;
The numerical model programme computes the demand displacement and angular movement (demand);

- The demand was fed into the DigiDAQ for logging. The demand was also transferred into the modified main actuator control software PACS (Packaged Actuator Control System, refer to De Catania et al. 2010 for details) and subsequently transferred to the Servo Controller (National Instrument NI7344-PCI);

- The demand (in analogue signal) was amplified through a linear power amplifier; and

- The VHM actuator moves with an inner control loop to achieve the demand $v$, $h$ and $\theta$ accurately.

The full control algorithm was programmed in LabVIEW (National-Instruments, 2003). The control panels are shown in Figure 6.16 and Figure 6.17. LabVIEW also allows programming of a wide range of mathematical operations. Therefore, the numerical programme was written in LabVIEW and was incorporated into the real-time control loop. This eliminated the communication between the numerical modelling and the real-time control algorithm, hence minimising any potential decay in the communication. The flight computer is permanently connected to the remote control PC in the control room. High speed communication between the two PC was achieved via optical fibre cables and slip rings which are housed within the main axis of the beam centrifuge. The test was controlled via the remote control PC and data was displayed in real time for monitoring. The maximum operating frequency of the control system is 100Hz.

Further details of the physical model and the numerical model are presented in Section 6.3.1 and Section 6.3.2 respectively. The loading system, which is the new VHM actuator, is described in detailed in Section 6.4.
Figure 6.14: Outline of Real-Time Hybrid Testing within a Geotechnical Centrifuge for the Modelling of Jack-up Reinstallation near Existing Footprints
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Figure 6.15: Connection between Real-time Control and Numerical Modelling

Figure 6.16: Modified Main Actuator Control Software PACS
6.3.1 Physical Model of the Jack-up Unit

The vertical, horizontal and moment loads were evaluated from axial gauges and bending gauges on the model jack-up leg. The measured loads were then converted to the $H_{CP}$, $V_{CP}$ and $M_{CP}$ at the common point. The sign convention and definition of terminologies adopted in the control algorithm are presented in Figure 6.18.
6.3.2 Numerical Model of Jack-up Unit

The components of the jack-up unit that were not included in the physical model were modelled numerically. In the numerical model, the jack-up unit and the two pre-embedded footings were simplified as shown in Figure 6.19. The numerical structure was assumed to be linear-elastic. It could therefore be reduced to a simple flexibility (inverse stiffness) matrix at the common point. The governing load-displacement relationship was in the general form presented in Figure 6.19. The movement at the common point was solved through a real-time stepping control algorithm.

Figure 6.18: Sign Convention and Definition of Terminologies
The flexibility matrix (C) described the load-displacement response of the rest of the jack-up unit and was pre-determined from separate structural analysis of the jack-up. The jack-up unit was simplified into a series of beam elements for analysis. The foundation was modelled as a series of linear and rotational springs. Structural analysis software SPACE-GASS (Integrated Technical Software Pty Ltd 2010) was used to analyse the load-displacement response at the common point. Unit horizontal, vertical and moment loads were applied separately at the common point, with the resultant displacement and angular movement at the common point divided by the unit load to determine the flexibility in the form of a 3x3 matrix.

In the simplified numerical model, the jack-up legs are connected to the hull with a fully-fixed node. Therefore, pure vertical load acting on the reinstalling jack-up leg (ΔV_{CP}) creates eccentric loading to the entire jack-up unit. If the pure vertical load at CP is fed into the numerical model, this would induce vertical, horizontal and angular movement at the CP. This is represented by the vertical load-displacement cross-coupling terms (C_{Vh}, C_{Vv} and C_{Vθ}) in the flexibility matrix. In reality, the jack-up leg was connected to the hull through a rack-and-pinion system. To prescribe pure vertical installation load without including the complex rack-and-pinion system in the numerical model, the installation process was simulated by a constant vertical displacement step ΔV_{step}. The vertical load-displacement cross-coupling terms (C_{Vh}, C_{Vv} and C_{Vθ}) in the flexibility matrix and ΔV_{CP} were therefore assumed to be zero. The simplified flexibility matrix is of the following form:
\[ \begin{pmatrix} \Delta h \\ \Delta v \\ \Delta \theta \end{pmatrix} = \begin{pmatrix} C_{hh} & 0 & C_{mh} \\ C_{hv} & 0 & C_{mv} \\ C_{h\theta} & 0 & C_{m\theta} \end{pmatrix} \begin{pmatrix} \Delta H_{cp} \\ 0 \\ \Delta M_{cp} \end{pmatrix} + \begin{pmatrix} 0 \\ \Delta v_{step} \\ 0 \end{pmatrix} \] (6.1)

In this study, the non-linear load-displacement response was ignored and the jack-up flexibility was assumed to be the same throughout reinstallation. The P-\(\Delta\) effect was therefore not considered. The change in jack-up flexibility is due to an increase in leg length with penetration depth, but this was also ignored. In theory, as the physical model leg penetrates into the soil, the length of the jack-up leg in the numerical model must increase by the same length. The common point (CP) of the jack-up model should shift down and the flexibility would therefore change with penetration. This concept is illustrated in Figure 6.20. In this study, the jack-up flexibility was developed assuming the footing at the top of seabed (depth=0). This assumption should not have a significant effect to the modelling of shallow depth reinstallation as the change in leg length was relatively small.

Figure 6.20: The Change of Jack-Up Leg Length and Shift of Model Common Point With Jack-Up Leg Penetration

The foundation response is in-elastic and provides different stiffnesses under loading and unloading conditions. The jack-up flexibility (which is a function of the jack-up structural properties and the foundation stiffness) is therefore different during loading and unloading conditions. This can be modelled by a simple linear-elastoplastic stiffness model as shown in Figure 6.21. The jack-up moves according to the \(C_{\text{unloading}}\) when subjected to an increase in load, a higher stiffness \(C_{\text{unloading}}\) should be adopted if
the loading is released. In this study, the worse case scenario was considered assuming zero \( C_{\text{unloading}} \). The model jack-up unit was rigid and could not move during unloading. This gave the largest movement during jack-up reinstallation.

![Figure 6.21: Linear-Elastoplastic Stiffness Model](image)

The main objective of this study was to investigate the effect of the structural properties to the jack-up reinstallation response. The above simplifications allowed the study to be conducted with a relatively simply numerical model and the test results provided valuable insights to understand the problem. The current numerical model could be modified in a future study for the modelling of more complex non-linear load-displacement responses.

### 6.3.3 Real-Time Control Algorithm

The basic real-time stepping control algorithm in Figure 6.6 was extended to incorporate the numerical model and the more specific operation required for the jack-up reinstallation test. The logic is presented in Figure 6.22.

The algorithm is triggered with the application of a constant vertical displacement \( \Delta V_{\text{step}} \) simulating the installation of the jack-up leg at a constant vertical rate (Step 1 of Figure 6.22). The loads acting on the physical jack-up model are measured by the strain gauges (\( V_{HM_i} \) in Step 2) and fed into the numerical model. The measured load is firstly converted to loads at CP (\( V_{HM_{CP_i}} \) in Step 3). The block average value, \( V_{HM_{\text{avei}}} \) is then evaluated to filter the noise in the measured data (Step 4). The block average \( V_{HM_{\text{avei}}} \) is then compared with the previous measured maximum load to determine where the physical jack-up leg is under a loading or unloading condition (Step 5, 6a, 6b). The demand displacement and angular movement is determined by solving the load-displacement relationship (Step 7). The demands are then sent to the actuators to be achieved within the next time-step. At the end of the time-step the loads and the
actuator position are measured and fed back into the numerical model, thus closing the loop (Step 9a). The loop is repeated until the physical model jack-up leg has reached the designed maximum penetration depth (Step 9b).

![Flow Chart of Combined Numerical Model and Real-time Control Algorithm](image)

The load readings from the bending gauges introduced considerable high frequency noise content to the signal being fed back into the numerical model. With the load-displacement relationship, this high frequency loading can result in high frequency
cyclic motion. This becomes of particular concern as the soil sample may be
undesirably remoulded at relatively shallow penetration where loading is small and the
movement was dominated by the noise. This problem, and a solution of mitigation by
using a filtering system, has been reported by, amongst others, Wagg et al. (2008). In
this study, the average loadings are evaluated from block of load readings of sample
size $F$, where $F$ is defined here as the number of load reading per blocks. The demand
displacement and angular movement is then calculated and implemented over the next
block time step $f$, where $f$ is defined as the time between the first and last reading of the
block of load reading ($f = F - 1$). The high frequency noise can effectively be filtered from
the system. The concept of block average loading is illustrated in Figure 6.23.

As discussed in Section 6.2.1, there are two prime sources of delay in the system, one
from the time-step delay in the feedback loop and the other from the inherent dynamics
of the actuators. Effort has been made to reduce the second source of delay. The
loading required to drive the scaled physical model is relatively small. Therefore, the
fast response servo-motor actuator was adopted instead of the hydraulic actuator. The
PID control (Proportional-Integral-Derivative, refer to Goodwin et al. 2002) of each
VHM actuator was tuned according to National-Instruments (2009) to minimise delay and
generate as accurate a motion as possible. The filter system introduces additional
delay into the loop. The load reading was stored until obtaining $F$ number of load
readings to calculate the demand displacement and angular movement for the next block
time step $f$. The block sample size $F$ was, therefore, carefully selected balancing the
delay and the noise level in the system.

Figure 6.23: Block Average of Loading (Modified after Blakeborough et al. 2001, with step
numbers from Figure 6.22)
6.4 VHM ACTUATOR

An actuator with three degrees-of-freedom was required to allow the jack-up model leg and spudcan to move independently in the vertical, horizontal and rotational direction. The existing linear actuator unit at UWA was adopted to provide the horizontal and vertical movement. An additional angular actuator, that can be attached to the existing linear actuator, was designed and built in this study. The integrated unit has been named the VHM actuator.

6.4.1 Existing Linear Actuator at UWA

The existing two degree-of-freedom actuator is shown in Figure 6.24. The tower of the vertical actuator sits in the frame of the horizontal actuator, forming a single linear motion actuator unit. The linear actuator unit is driven by a separate harmonic motor through a geared ball screw, allowing independent or combined linear motions. Further details on the linear actuator unit are provided in Randolph et al. (1991).

Figure 6.24: Existing Linear Actuator at UWA (Arrows Indicate Movement Directions)
6.4.2 Design of an Angular Actuator

A summary of existing VHM actuators is presented in Table 6.2. The first VHM loading system was developed by Martin (1994) to operate in the 1g environment (Figure 6.25). The apparatus uses three digitally controlled stepper motors allowing independent vertical, horizontal and rotational movement. Rotation is achieved through a rack and pinion arrangement set at a fixed radius of 425mm. It is mounted on the horizontal motion system, which in turn is mounted on the vertical motion system. The system was developed with the prime objective to establish the failure envelope of a spudcan under combined load. Therefore, the pivot point of rotation was fixed at the spudcan load reference point.

<table>
<thead>
<tr>
<th>Table 6.2: Comparison of Angular Movement Actuators</th>
</tr>
</thead>
<tbody>
<tr>
<td>-------------</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Mechanical configuration</td>
</tr>
<tr>
<td>Maximum Moment (Nm)</td>
</tr>
<tr>
<td>Maximum Operation Acceleration</td>
</tr>
<tr>
<td>Maximum Rotation Angle (°)</td>
</tr>
<tr>
<td>Maximum Rotation Rate (°/s)</td>
</tr>
</tbody>
</table>

Byrne and Houlsby (2005) developed a Stewart Platform (Stewart 1965) loading system to conduct foundation tests with six degree-of-freedom movement (6-dof). Six carefully aligned actuators are controlled to achieve 6-dof movement. As shown in Figure 6.26, three actuators were approximately vertical and three others were approximately horizontal. To achieve movement in the purely vertical direction, the three vertical actuators must move the same distance and the horizontal actuators must also move slightly to adjust the offset in the horizontal direction. Bienen et al. (2005) employed the loading system to generate a maximum angular movement of 5° at 0.01°/s.
VHM actuators have also been developed for application in geotechnical centrifuges. Dean et al. (1997) provided rotational movement by driving a pair of horizontal actuators (Figure 6.27). An advantage of this configuration is that by driving the two horizontal actuators to different displacement, the pivot point of rotation can be varied. Punrattansin et al. (2003) developed a similar system for a beam centrifuge but instead used a pair of vertical loading jacks. Figure 6.28 shows that by applying different vertical displacements in the two jacks, rotational movement could be generated at any point in the model. However, the movements from such a paired or multi-actuator system are not linearly or independently related to the motion of each actuator. More complex control routines are required to achieve the desired movements.

Figure 6.25: The VHM Loading System Developed by Martin (1994)
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Figure 6.26: Six Degree-Of-Freedom Loading System Developed by Byrne and Houlsby (2005)

Figure 6.27: VHM Loading System for Testing in Centrifuge Facilities Developed by Dean et al. (1997)
In designing a VHM actuator for the UWA beam centrifuge two major constraints were identified: the actuator (1) must be accommodated within the limited space of the beam centrifuge and (2) be able to operate under the large working loads in a high gravitational environment. An efficient option saw integration of the angular actuator within the existing and proven linear actuator. The existing linear actuator unit also provided robust structural support to the angular actuator in the high gravity environment, though loss of vertical and horizontal travel distance had to be accommodated.

The limited space in the existing linear actuator unit precluded the use of a pair of actuators to generate the angular movement (such as used by Dean et al. 1997, Punrattansin et al. 2003). A rack-and-pinion system on the other hand requires relatively limited space. However, the precision of any such system is limited by the finite increment of each tooth. Therefore, an innovative “Pulley System” was developed in this study. As shown in Figure 6.29, the rotational motion from the motor was converted to angular motion at a pivot point through a pulley system. The main advantage of this system is that there is only one pivot point in the system and the single motor module was aligned in the line of action. This generates precise angular movement from a relatively slim configuration.
Another major advantage of the VHM actuator is that the centre of rotation can be prescribed anywhere along the vertical axis. This concept is illustrated with the example shown in Figure 6.30. When the angular actuator rotates around the pivot point ($\Delta \theta$), the linear actuators can also move the pivot point in the vertical and horizontal direction ($\Delta x$ and $\Delta y$), offsetting the linear movement so that there is pure rotational movement at any prescribed point along the vertical axis.
6.4.3 Specification of the Angular Actuator

The angular actuator was designed to provide accurate angular movements within the angular stroke under the design working load. Table 6.3 shows the general specification of the VHM actuator and the VHM actuator unit is presented in Figure 6.31. The design requirements and the design capacities of the angular actuator are summarised in Table 6.4 and Table 6.5.

Table 6.3: General Specification of the VHM Actuator

<table>
<thead>
<tr>
<th></th>
<th>Vertical</th>
<th>Horizontal</th>
<th>Angular</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum g</td>
<td>250g(^1)</td>
<td></td>
<td>100g(^2)</td>
</tr>
<tr>
<td>Maximum Stroke</td>
<td>180mm</td>
<td>70mm</td>
<td>± 3°</td>
</tr>
<tr>
<td>Maximum Speed</td>
<td>3mm/s</td>
<td>30mm/s</td>
<td>3.96°/s</td>
</tr>
<tr>
<td>Maximum Force/Moment</td>
<td>(V_{\text{max}} = 6500\text{N})</td>
<td>(H_{\text{max}} = 2000\text{N})</td>
<td>(M_{\text{max}} = 180\text{Nm @ Pivot Point})</td>
</tr>
<tr>
<td>Movement Precision</td>
<td>2.84E-5mm/count</td>
<td>2.84E-5mm/count</td>
<td>1.49E-5 °/count</td>
</tr>
</tbody>
</table>

1. The original actuator is designed to operate to 250g
2. The angular actuator is designed to operate up to 100g

Figure 6.30: Example of Shifting the Centre of Rotation from Actuator Pivot Point to Model Footing Level
The angular actuator was designed to resist the vertical, horizontal and moment loads generated from the jack-up leg penetrating and extracting from the soil sample. The maximum working loads $H_{LRP}$ and $M_{LRP}$ were determined from previous centrifuge
modelling test results published by Cassidy et al. (2009) and Gaudin et al. (2007). The maximum design load and the capacity of the critical elements: the pivot point, the cables and the motor are presented in Table 6.4. The detailed calculations are presented in Appendix F.1.

The movement criteria are listed in Table 6.5. A jack-up unit in practice would usually tolerant tilting at platform level of up to $2^\circ$ before resulting damage in the leg-hull connection (Purwana 2010). In this study it was conservatively assumed that all the tilting at platform level was resulted from angular movement of jack-up leg installed nearby footprint. The angular actuator was designed to have a minimum angular stroke $2^\circ$ with two degree of freedom accommodating both clockwise and anti-clockwise movement. The detailed calculations of the speed and precision requirements are presented in Appendix F.2.

<table>
<thead>
<tr>
<th>Design Requirement</th>
<th>Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angular Stroke</td>
<td>+/-2°</td>
</tr>
<tr>
<td></td>
<td>+/-3°</td>
</tr>
<tr>
<td>Angular Speed</td>
<td>0.03°/s</td>
</tr>
<tr>
<td></td>
<td>3.96°/s</td>
</tr>
<tr>
<td>Precision (Motor Driving)</td>
<td>In the order of 1.4E-5°/count</td>
</tr>
<tr>
<td></td>
<td>1.49E-5°/count</td>
</tr>
<tr>
<td>Precision (Front Encoder)</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>4.5E-3 °/count</td>
</tr>
</tbody>
</table>

Figure 6.32 shows the angular actuator with the key components: the motor module, the cable & pulley, the pivot point and the actuator box. The motor module, the encoders and the bearing systems were purchased from specialist suppliers. The actuator was fabricated and assembled at the Civil Engineering Workshop of UWA. The new angular actuator has an outer dimension of $100\text{mm} \times 79\text{mm} \times 411\text{mm}$ (Width$\times$Thickness$\times$Height) and weights 2.57kg. After accommodating the angular actuator, the stroke in the linear actuators reduced to 180mm (vertical) and 70mm (horizontal). A detailed discussion on the design and fabrication of the key components is presented in Appendix F3 & F4.
There are two prime sources of error in the system: the motor back-lash and the cable extension. The theoretical error magnitudes were 1% and 3% respectively. The angular actuator was carefully calibrated to verify the magnitude of error. It was found that the error was within 1% of prescribed angular movement and was considered to be acceptable. The detailed calibration procedures are presented in Appendix F.5. Figure 6.33 shows the VHM actuator in position for a test.
Figure 6.33: The VHM Actuator in Position for a Test

6.5 VALIDATION OF THE REAL-TIME HYBRID TESTING SYSTEM

The real-time hybrid testing method and system was validated by installing a physical jack-up model in compressible foam material (Figure 6.34). The control algorithm was operating at one load reading every 0.01s (100Hz). The average load reading was calculated over each 10 data, corresponding to a block sample size of F=10 and the frequency of load-displacement responses was therefore 10Hz. The delay due to the filtering system is assessed in the following sections.
In the first stage, the cross-coupling terms in the flexibility matrices were not considered. This simplified the relationship of Equation (6.2) and (6.3).

$$\Delta h = C_{Hh} x \Delta H_{CP} \quad (6.2)$$

$$\Delta \theta = C_{M\theta} x \Delta M_{CP} \quad (6.3)$$

A range of horizontal flexibility ($C_{Hh}$) and rotational flexibility ($C_{M\theta}$) were considered in isolation to check the horizontal and rotational axes separately. The list of validation tests is shown in Table 6.6. In each test, the corresponding $C_{\text{loading}}$ and $C_{\text{unloading}}$ was input into the real-time hybrid testing control panel. The physical jack-up leg was then penetrated with half of the footing in contact with the foam, generating vertical, horizontal and moment loads.
Table 6.6: List of Validation Test

<table>
<thead>
<tr>
<th>Validation Test Name</th>
<th>$C_{\text{loading}}$</th>
<th>$C_{\text{unloading}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>VT1</td>
<td>$C_{\text{th}} = 1.93$</td>
<td>0</td>
</tr>
<tr>
<td>VT2</td>
<td>$C_{\text{th}} = 0.38$</td>
<td>0</td>
</tr>
<tr>
<td>VT3</td>
<td>$C_{\text{th}} = 0.23$</td>
<td>0</td>
</tr>
<tr>
<td>VT4</td>
<td>$C_{\text{M0}} = 1.94$</td>
<td>0</td>
</tr>
<tr>
<td>VT5</td>
<td>$C_{\text{M0}} = 0.36$</td>
<td>0</td>
</tr>
<tr>
<td>VT6</td>
<td>$C_{\text{M0}} = 0.21$</td>
<td>0</td>
</tr>
</tbody>
</table>
| VT7                  | \[
C = \begin{bmatrix}
1.93 & 0 & -6.62 \\
2.22 & 0 & -11.01 \\
-0.38 & 0 & 1.94 \\
\end{bmatrix} \]

The horizontal and angular movement ($h$ & $\theta$) from the actuator encoders was plotted against the horizontal force and moment measured from the strain-gauges of the physical jack-up leg model, as shown in Figure 6.35. The theoretical load-displacement relationship graphs were determined by multiplying the flexibility with the measured loads. It was found that the two graphs are identical, confirming the input flexibility has been implemented correctly and the two actuator axes can generate accurate movement.

![Figure 6.35: LHS-Validation of Horizontal Load-Displacement Response; RHS-Validation of Moment-Rotational Response](image-url)
A further validation test was conducted for the full C matrix that allows cross-coupling between vertical, horizontal and rotational responses. The results are presented in Figure 6.36. It was found that the measured load-displacement responses are in good agreement with the theoretical responses of the jack-up unit. In order to assess the effect of delay, the same results were plotted against the theoretical responses adjusted by one block time step (f) delay and the results are shown in Figure 6.37. The agreement improved significantly. Therefore, the delay due to the filtering system was the major contribution factor to the very small difference observed between the measured and the theoretical load-displacement responses. As demonstrated above, the system can accurately model the responses of a jack-up unit in a real-time environment.
Chapter 6: Development of a Real-time Hybrid Testing Method

Figure 6.37: Load Displacement Responses of Full C matrix – Theoretical Responses Adjusted for Delay Due to Filtering

6.6 CONCLUDING REMARKS

The real-time hybrid testing method pioneered in structural dynamic testing has been extended to a geotechnical application. A method for investigating the reinstallation of a jack-up unit near a footprint has been detailed in this chapter. A new VHM actuator was required and a full explanation of the design criteria, setup and validation has been provided in this chapter. Validation tests have been conducted demonstrating that good quality test data can be obtained. The delay in the system was also kept within acceptable limits. This testing method, apparatus and system were used in a parametric study on the effect of jack-up structural stiffness to the response of jack-up reinstallation, as is presented in the next two chapters.
CHAPTER 7. COMBINED EFFECT OF FOOTPRINT GEOMETRY AND JACK-UP STRUCTURAL PROPERTIES ON REINSTALLATION

7.1 INTRODUCTION

The real-time hybrid testing method developed in Chapter 6 was adopted here to investigate the effect of structural properties to jack-up reinstallation response. This chapter presents the investigation on the combined effect of footprint geometry and jack-up structural properties (Figure 7.1). An idealised footprint cavity was considered to eliminate the effect of soil heterogeneity in the footprint jack-up interaction.

In this chapter, the jack-up model and testing apparatus are first introduced, followed by a detailed description on the testing procedures and programmes. The results of reinstallation tests with jack-up units of different flexibility are then presented. Finally, the change in reinstallation response with change in jack-up structural properties is discussed.

Figure 7.1: The Two Governing Parameters Modelled by the Scaled Model Tests

7.2 JACK-UP MODELS

Jack-up units are unique in that they have different leg lengths, leg spacing, spudcan sizes and so on. Figure 7.2 shows the dimension and the key structural properties of the jack-up model. With the consideration on the maximum number of test to be conducted within a soil sample and the correctness of scaled spudcan size, the diameter of spudcan
adopted in this model was 6m, which is around 3 times smaller than the typical dimension of a spudcan used in practice. The moment acting on the testing spudcan would therefore be 27 times smaller than on an 18m spudcan (with moment proportional to the cubic of footing diameter). Since the main purpose of this study is to investigate the load-displacement responses during jack-up reinstallation, the leg flexural rigidity was also scaled down proportionally to maintain the bending properties. The leg flexural rigidity adopted was of a similar reduced ratio, with $\text{EI}=13.3\times10^6 \text{kNm}^2$, around 5-100 times lower than the range of values reported in other publications (e.g. $\text{EI}=71.2\times10^6 \text{kNm}^2$ and $\text{EI}=1440\times10^6 \text{kNm}^2$ in Murff et al. (1991) and Vlahos (2004), respectively).

**Figure 7.2: Dimension and Properties of Model Jack-Up Unit**

<table>
<thead>
<tr>
<th>Jack-up Properties:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s Modulus E</td>
<td>200GPa</td>
</tr>
<tr>
<td>Shear Modulus G</td>
<td>8000MPa</td>
</tr>
<tr>
<td>Poisson Ratio</td>
<td>0.25</td>
</tr>
</tbody>
</table>
Table 7.1: Prototype Dimension of Properties of Jack-up Unit Models

<table>
<thead>
<tr>
<th>Prototype Dimension</th>
<th>This Study</th>
</tr>
</thead>
<tbody>
<tr>
<td>Leg Length, L (m)</td>
<td>94</td>
</tr>
<tr>
<td>Leg Spacing, W (m)</td>
<td>60</td>
</tr>
<tr>
<td>Leg Flexural Rigidity, EI (kNm²)</td>
<td>13.3E+6</td>
</tr>
<tr>
<td>Cross-Sectional Area of Leg, A (m²)</td>
<td>0.25</td>
</tr>
<tr>
<td>Diameter of Spudcan, D (m)</td>
<td>6</td>
</tr>
</tbody>
</table>

7.2.1 Numerical Jack-up Model

As described in Chapter 6, while a section of jack-up leg (23.5 m long section) located near the footprint was modelled physically, the remainder of the jack-up unit was simplified and modelled numerically. Figure 7.2 shows the simplified numerical model. Each jack-up leg lattice was simplified and modelled as a beam element with equivalent bending stiffness. The hull of each jack-up unit was modelled as three beam elements in a simple equilateral triangle. A finite stiffness was prescribed for the hull element to allow bending that could occur in practice. The equivalent dimensions of each hull element were determined as a ratio with respect to the dimensions of the leg elements. This approach was also adopted in Martin and Houlsby (1999). The leg-hull connection was simplified and modelled as fully-fixed nodes.

For the scaled model test at unit gravity, a 1:100 numerical model was established. The prototype dimensions were scaled down according to the scaling factors presented in Table 3.1.

During jack-up reinstallation, the two jack-up legs located further away from the footprint will be installed first gaining some foundation fixity. The foundation fixity will therefore contribute to the structural properties of jack-up unit. The two jack-up footings embedded in soil were modelled as a series of linear and rotational springs. The spring stiffness, defined by the vertical stiffness \( K_v \), the horizontal stiffness \( K_h \) and the rotational stiffness \( K_{rot} \) were evaluated based on Boussinesq elastic solutions. The depth factor, defined by \( k_{dv} \), \( k_{dh} \) and \( k_{drot} \), was applied according to SNAME (2002) to incorporate the footing embedment depth. The footing embedment depth ratio \( z/D \) (depth of footing over diameter of footing) was assumed to be 0.25. This is a conservative assumption as the embedment depth of the two footings varies greatly depending on the site condition and the embedment depth can be up to 2D in practice. The equations are presented in (7.1), (7.2) and (7.3), where \( \nu \) is the Poisson ratio, \( G \) is the shear modulus of the soil and \( D \) is the diameter of the footing.
Chapter 7: Combined Effect of Footprint Geometry and Jack-up Structural Properties on Reinstallation

\[ K_v = k_{dv} \frac{4GD}{2(1-v)} \quad (7.1) \]

\[ K_h = k_{dh} \frac{32(1-v)GD}{2(7-8v)} \quad (7.2) \]

\[ K_{rot} = k_{drot} \frac{8GD^3}{6(1-v)} \quad (7.3) \]

G was estimated by the direct proportional relationship between rigidity index \( I_r \) and undrained shear strength \( s_u \) (Equation (7.4)). \( I_r \) was determined based on Cassidy et al. (2004), as represented in the latest version of the SNAME guidelines (Equation (7.5)).

\[ G = I_r s_u \quad (7.4) \]

\[ I_r = \frac{600}{OCR^{0.25}} \quad (7.5) \]

The foundation stiffness values, namely, \( F_{SNAME} \), are presented in Table 7.2. This was assumed as the base case of foundation stiffness. In order to capture a range of movement, an upper bound corresponding to an encastré boundary condition of infinite foundation stiffness (\( F_{INFINITE} \)) and a lower bound assuming one-tenth of the SNAME foundation stiffness (\( F_{0.1xSNAME} \)) were also considered.

**Table 7.2: Soil Properties and Foundation Stiffness Values (\( F_{SNAME} \))**

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Scaled Model Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>D</td>
<td>0.06m</td>
</tr>
<tr>
<td>( kD/s_{sum} )</td>
<td>0.22</td>
</tr>
<tr>
<td>( s_u ) at ( z/D=0.25 )</td>
<td>5.00kPa</td>
</tr>
<tr>
<td>( OCR ) at ( z/D=0.25 )</td>
<td>3000</td>
</tr>
<tr>
<td>( K_v )</td>
<td>111.88kN/m</td>
</tr>
<tr>
<td>( K_h )</td>
<td>86.26kN/m</td>
</tr>
<tr>
<td>( K_{rot} )</td>
<td>0.07kNm/rad</td>
</tr>
</tbody>
</table>

The physical and numerical jack-up models connect to each other at the common point, CP, forming the complete jack-up model (refer to Section 6.3.2 for details). The load-displacement response of each numerical jack-up unit was analysed using structural analysis software SPACE-GASS (Integrated Technical Software Pty Ltd 2010). A unit horizontal force, vertical force and moment were applied separately at the CP; each resulting movement at the common point was divided by the unit load to determine the flexibility in the form of a 3x3 matrix (Equation (7.6)). The flexibility matrices adopted in the scaled model tests are presented in Table 7.3.
Chapter 7: Combined Effect of Footprint Geometry and Jack-up Structural Properties on Reinstallation

\[ C = \begin{pmatrix} C_{Hh} & 0 & C_{Mh} \\ C_{Hv} & 0 & C_{Mh} \\ C_{H0} & 0 & C_{Mh} \end{pmatrix} \]  

(7.6)

Table 7.3: Flexibility Matrices of Jack-up Model for Scaled Model Tests

| Flexibility Matrices for Scaled Model Test (mm/N, mm/Nm, deg/N or deg/Nm) |
|-----------------|-----------------|
| C-F*FINITE      |                 |
| 1.49E+00        | 0               | -2.93E+00 |
| -4.87E-01       | 0               | -1.94E+00 |
| -1.68E-01       | 0               | 5.32E+01  |

| C-F*RANDOM      |                 |
| 5.49E+00        | 0               | 1.39E+01  |
| 8.24E+00        | 0               | -3.91E+01 |
| 7.94E+01        | 0               | 4.62E+00  |

7.2.2 Modifications to Flexibility Matrices

The full flexibility matrices were modified to investigate the effect of structural properties in three different phases of testing (Figure 7.3):

Phase 1: A simple single axis linear load-displacement relation, either horizontal or rotational was considered (Figure 7.3a). This modelled a jack-up leg connecting to the actuator with a horizontal spring or hinge connection.

Phase 2: A linearly combined horizontal and rotational flexibility was considered (Figure 7.3b). The jack-up unit was allowed to slide according to the horizontal load-displacement flexibility \( C_{Hh} \) if subjected to horizontal force. At the same time, the jack-up unit could also rotate according to \( C_{M0} \) in response to the moment acting on the jack-up unit. This modelled an idealised jack-up with independent horizontal and moment load-displacement responses.

Phase 3: A flexibility matrix with horizontal and angular load-displacement cross-coupling terms were adopted (Figure 7.3c). With the horizontal force-rotation cross-coupling terms \( C_{H0} \), horizontal force acting on the jack-up unit induces rotation of the jack-up. Similarly, moment acting in the jack-up unit could induce horizontal movement according to the moment-horizontal displacement coupling terms \( C_{Mh} \). This relatively complex load-displacement relation allowed the modelling of more realistic jack-up reinstallation response.

In all three testing phases, the vertical load-displacement cross-coupling terms \( C_{Vh}, C_{VV} \) and \( C_{V0} \) were assumed to be zero. The vertical installation of the jack-up leg was
modelled by the application of a constant vertical displacement step $\Delta v_{\text{step}}$. Detailed discussions on the assumptions and simplifications on the numerical model are presented in Section 6.3.2.

<table>
<thead>
<tr>
<th>Phase</th>
<th>Flexibility Matrix</th>
</tr>
</thead>
</table>
| 1     | Horizontal Flexibility Only:  
\[
\begin{pmatrix}
\Delta h \\
\Delta v \\
\Delta \theta
\end{pmatrix} =
\begin{pmatrix}
C_{\text{Hh}} & 0 & 0 \\
0 & 0 & 0 \\
0 & 0 & 0
\end{pmatrix}
\begin{pmatrix}
\Delta H_{\text{CP}} \\
\Delta \theta
\end{pmatrix} +
\begin{pmatrix}
\Delta v_{\text{step}} \\
0
\end{pmatrix}
\]  
  (a): Phase 1 - Single Axis Linear Flexibility Matrix |
| 2     | Rotational Flexibility Only:  
\[
\begin{pmatrix}
\Delta h \\
\Delta v \\
\Delta \theta
\end{pmatrix} =
\begin{pmatrix}
0 & 0 & 0 \\
0 & 0 & 0 \\
0 & 0 & C_{\text{Mj}}
\end{pmatrix}
\begin{pmatrix}
\Delta H_{\text{CP}} \\
\Delta M_{\text{CP}}
\end{pmatrix} +
\begin{pmatrix}
\Delta v_{\text{step}} \\
0
\end{pmatrix}
\]  
  (b) Phase 2 - Linearly Combined Horizontal and Rotational Flexibility Matrix |
| 3     | Cross-Coupled Horizontal and Rotational Flexibility Matrix:  
\[
\begin{pmatrix}
\Delta h \\
\Delta v \\
\Delta \theta
\end{pmatrix} =
\begin{pmatrix}
C_{\text{Hh}} & C_{\text{Mh}} \\
0 & 0 & 0 \\
C_{\text{Hj}} & C_{\text{Mj}}
\end{pmatrix}
\begin{pmatrix}
\Delta H_{\text{CP}} \\
\Delta M_{\text{CP}}
\end{pmatrix} +
\begin{pmatrix}
\Delta v_{\text{step}} \\
0
\end{pmatrix}
\]  
  (c) Phase 3 - Cross-Coupled Horizontal and Rotational Flexibility Matrix |

Figure 7.3: Modified Flexibility Matrices in Three Testing Phases
7.2.3 Physical Jack-up Model

Figure 7.4 shows the 1:100 scaled model of the 23.5m long section of jack-up leg and footing used in the real-time hybrid testing. The model footing was made of solid aluminium and the footing has a flat-base to allow for full contact with the flat surface of the soil sample at the touchdown level. The model jack-up leg was fabricated from an aluminium tube. Its properties are presented in Table 7.4. The scaling factors are also shown in Table 7.4 for ease of reference. The model jack-up leg was 235mm long, enabling it to fit into the limited headroom in the beam centrifuge facility. The diameter and the wall thickness were designed to model the bending stiffness of the prototype jack-up unit. The wall thickness was locally reduced at the locations of the axial and bending gauges. This allowed larger strains to be generated and improved the sensitivity of measurements without scarifying the overall bending stiffness of the physical jack-up leg model.

![Model Jack-up Leg and Footing Used in Real-time Hybrid Testing (All Dimensions in mm)](image-url)
Table 7.4: Prototype and Model Properties of Jack-Up Leg

<table>
<thead>
<tr>
<th></th>
<th>Prototype</th>
<th>Scaling Factor</th>
<th>Model @ 100g</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cross-sectional Area, A</td>
<td>0.25m²</td>
<td>-</td>
<td>72.4mm²</td>
</tr>
<tr>
<td>Outer Diameter</td>
<td>-</td>
<td>-</td>
<td>16.0mm</td>
</tr>
<tr>
<td>Thickness</td>
<td>-</td>
<td>-</td>
<td>12.8mm</td>
</tr>
<tr>
<td>L</td>
<td>23.5m</td>
<td>N</td>
<td>235mm</td>
</tr>
<tr>
<td>EI/L³</td>
<td>1.01E+3 kN/m</td>
<td>N</td>
<td>10.1kN/m</td>
</tr>
</tbody>
</table>

7.3 ASSUMPTIONS AND LIMITATIONS OF THE SCALED MODEL TEST AT UNIT GRAVITY

The combined effect of footprint geometry and jack-up structural properties to the reinstallation response was investigated using a scaled model test at unit gravity. While centrifuge modelling allows realistic simulations of foundation response with stress level in the soil comparable to the prototype, reasonable modelling accuracy can still be achieved using scaled model tests at unit gravity for undrained geotechnical problems (Martin and Houlsby 2000). This is because the shallow foundation capacity in fine grained soils depends primarily on soil undrained shear strength rather than on gravitational forces. 1g-scaled model test is therefore a cost-effective and convenient alternative for parametric studies. Furthermore, the plastic flow of the soil during footing penetration is not affected by the gravity (Martin and Houlsby 2000). However, a 1g-scaled model test is unable to model the following two aspects of the jack-up reinstallation problem:

1) Backfilling of the cavity. When the footing penetrates deeper into the soil, Hossain (2008) demonstrated a stable cavity will be formed, with further penetration, the weaker soil above the footing level will backfill into the cavity due to deep flow around the failure mechanism. This phenomenon cannot be reproduced in scaled model tests at unit gravity. While the determined vertical bearing capacity will be higher without backfilling, the horizontal and moment capacities will be lower. However, this is more critical to the jack-up reinstallation near the real footprint than near the idealised footprint cavity. The real footprint, with realistic geometry and strength variation, can only be created with a real soil failure mechanism. The accuracy of a jack-up reinstallation near an idealised footprint cavity depends on correct geometry and undisturbed soil around the footing, both of which are easily achieved for both unit gravity and high gravity conditions. Secondly, backfilling of the cavity was not expected within the critical depth where peak loadings occur. As discussed in Chapter 3,
the peak loads occurred within the depth of the footprint toe, typically 0.33D. This depth is well above the critical cavity height (0.54D) for the soil sample considered in this study.

2) Overburden pressure. The surcharge pressure from overburden might reduce the vertical, horizontal and moment bearing capacity of the footing. The lower self-weight in scaled model test condition gives a lower overburden pressure when compared to the field condition and the difference increases with depth. However, the effect of the difference in overburden pressure is less critical during shallow depth response (which is the main concern of footprint geometry).

The scaled model test at unit gravity offers two advantages. First, it allows detailed observations of the sliding and rotation failure of the physical jack-up model throughout the test. Second, it has the advantage of a low noise level in the strain-gauge on the physical jack-up leg model. The noise level is typically within 0.01N in horizontal force and 0.001Nm in moment during scaled model test at unit gravity, but it is amplified to 0.5N and 0.02Nm (model scale) in the beam centrifuge at 100g. When the higher quality load data was fed into the control algorithm and numerical modelling, better quality test results were obtained. Thus, the advantages of the scaled model test at unit gravity outweighed the shortcomings, and the 1g-scaled model test was adopted for the parametric study of the isolated structural effects of jack-up reinstallation.

7.3.1 Scaled Model Test at Unit Gravity

Scaled model testing was conducted on the laboratory floor in the Civil Engineering Laboratory at UWA. The setup of the testing bay mirrors the beam centrifuge facility. The power amplifier, the servo-controller and the digiDAQ box are identical to those used in the centrifuge facility except that the control and monitoring PC and the flight PC are merged into one unit. The real-time hybrid testing algorithm is explained in Chapter 6. The real-time hybrid scaled model test at unit gravity is referred to as 1g-scaled model test for the reminder of this thesis.

7.3.2 Preparation of Soil Sample and Idealised Footprint Cavity

The soil sample was prepared and tested in a strongbox. The details of the strongbox and the preparation of kaolin slurry are presented in Section 4.4.1. The kaolin slurry was consolidated with a maximum consolidation pressure of 300kPa in the consolidation press. This method produced heavily over-consolidated clay. The undrained shear strength profile within the critical soil depth, which is within the footprint toe level (z/D=0.33), was comparable to the undrained shear strength profile of
the slightly over-consolidated clay sample adopted in the beam centrifuge tests, as will be discussed further in Section 8.4.1.

After removal from the consolidation press, the soil sample was left to swell for seven days before testing. The final soil sample height was approximately 210mm. A series of t-bar characterisation tests were conducted at different times to confirm the completion of swelling.

The footprint cutting apparatus shown in Figure 3.10 was modified slightly for preparation of footings in the strongbox (Figure 7.5). The idealised footprint cavity was then prepared according to Section 3.3.6. Only one footprint geometry, footprint TB, which was the one most aligned with the real footprint geometry, was considered. Its dimensions are provided in Table 3.6.

![Figure 7.5: Preparation of Idealised Footprint Cavity in Strongbox](image)

7.3.3 Testing Procedures of Scaled Model Test at Unit Gravity

After the idealised footprint cavity was created, the strongbox was filled with water for testing to ensure full saturation of the soil sample. The real-time hybrid testing control programme was launched and the numerical jack-up model was established by inputting the \( C_{\text{loading}} \) and \( C_{\text{unloading}} \) matrices into the programme. The VHM actuator and the physical jack-up model were mounted on the strongbox. The centre of the footing was at 1.0D offset from an idealised footprint cavity. The test started by penetrating the physical jack-up model at 0.1mm/s into the soil. The vertical force, horizontal force, moment, vertical displacement, horizontal displacement and rotation angle were monitored throughout the test. After reaching the maximum depth of 100mm below the soil surface, the physical jack-up model was then extracted and moved to another test site for subsequent testing.
7.3.4 List of Scaled Model Tests at Unit Gravity and As-built Layout of Test Sites

In total, ten 1g-scaled model tests were conducted in four boxes of soil samples. All the reinstallation tests were conducted at 1.0D offset from the centre of idealised footprint cavities. The only variable in the experimental investigation was the structural properties. Different jack-up structural properties with different foundation stiffness or different flexibility type at the CP were considered and a summary is presented in Table 7.5. The nomenclature of the tests is in the form of “test type-flexibility matrix type-test number”. For the first letter, S stands for 1g-scaled model test. For the different flexibility type adopted, the nomenclature is as follows:

- R for zero flexibility as the model jack-up leg was fully-fixed to the actuator;
- H for horizontal flexibility only;
- M for rotational flexibility only;
- L for linearly combined horizontal and rotational flexibility;
- C for cross-coupled horizontal and rotational flexibility.

The test number refers to the number of similar tests conducted in the series. In each soil sample, a reinstallation test was conducted with the model leg fully-fixed to the actuator connection and the model jack-up leg could only move vertically into the soil. This is referred to as rigid reinstallation in subsequent discussions. This rigidity is different from the foundation rigidity defined by $F_{\infty}$. $F_{\infty}$ models two pre-embedded footings rigidly connected to the ground. The reinstalling leg could still slide or rotate during reinstallation because the jack-up itself has finite flexibility.
The typical testing layout is presented in Figure 7.7 and Figure 7.8. The minimum distance between a test site and the nearest boundary was kept to 1.5D to minimise the boundary effect. To minimise the effect of soil disturbance on subsequent tests, the minimum distance between test sites was 3.0D (from the footing centre). Based on visual inspection, the minimum distance of 3.0D was sufficient as the zone of soil disturbance after the completion of the jack-up reinstallation test was typically within 2.0D.

Table 7.5: List of 1g- Scaled Model Tests

<table>
<thead>
<tr>
<th>Test Name</th>
<th>Test Phase</th>
<th>H Flexibility</th>
<th>M Flexibility</th>
<th>Modified Flexibility Matrix</th>
<th>Foundation Stiffness</th>
<th>Sample Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-R-1</td>
<td></td>
<td>-</td>
<td>-</td>
<td>( c = \begin{pmatrix} 0 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 0 \end{pmatrix} )</td>
<td>Rigid</td>
<td>B1</td>
</tr>
<tr>
<td>S-R-2</td>
<td></td>
<td>-</td>
<td>-</td>
<td>( c = \begin{pmatrix} 1.49 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 0 \end{pmatrix} )</td>
<td>( F_{\infty} )</td>
<td>B2</td>
</tr>
<tr>
<td>S-R-3</td>
<td></td>
<td>-</td>
<td>-</td>
<td>( c = \begin{pmatrix} 5.49 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 0 \end{pmatrix} )</td>
<td>( F_{0.1xSNAME} )</td>
<td>B3</td>
</tr>
<tr>
<td>S-R-4</td>
<td></td>
<td>-</td>
<td>-</td>
<td>( c = \begin{pmatrix} 0 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 0.532 \end{pmatrix} )</td>
<td>( F_{\infty} )</td>
<td>B4</td>
</tr>
<tr>
<td>S-H-1</td>
<td>1</td>
<td>✓</td>
<td>-</td>
<td>( c = \begin{pmatrix} 1.49 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 0 \end{pmatrix} )</td>
<td>( F_{\infty} )</td>
<td>B1</td>
</tr>
<tr>
<td>S-H-2</td>
<td>1</td>
<td>✓</td>
<td>-</td>
<td>( c = \begin{pmatrix} 5.49 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 0 \end{pmatrix} )</td>
<td>( F_{0.1xSNAME} )</td>
<td></td>
</tr>
<tr>
<td>S-M-1</td>
<td>1</td>
<td>-</td>
<td>✓</td>
<td>( c = \begin{pmatrix} 0 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 0.532 \end{pmatrix} )</td>
<td>( F_{\infty} )</td>
<td>B2</td>
</tr>
<tr>
<td>S-M-2</td>
<td>1</td>
<td>-</td>
<td>✓</td>
<td>( c = \begin{pmatrix} 0 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 4.620 \end{pmatrix} )</td>
<td>( F_{0.1xSNAME} )</td>
<td></td>
</tr>
<tr>
<td>S-L-1</td>
<td>2</td>
<td>✓</td>
<td>✓</td>
<td>( c = \begin{pmatrix} 1.49 &amp; 0 &amp; -2.930 \ 0 &amp; 0 &amp; 0 \ -0.168 &amp; 0 &amp; 0.532 \end{pmatrix} )</td>
<td>( F_{\infty} )</td>
<td>B3</td>
</tr>
<tr>
<td>S-C-1</td>
<td>3</td>
<td>✓</td>
<td>✓</td>
<td>( c = \begin{pmatrix} 1.49 &amp; 0 &amp; -2.930 \ 0 &amp; 0 &amp; 0 \ -0.168 &amp; 0 &amp; 0.532 \end{pmatrix} )</td>
<td>( F_{\infty} )</td>
<td>B4</td>
</tr>
</tbody>
</table>
7.4 SCALED MODEL TEST AT UNIT GRAVITY - EXPERIMENTAL RESULTS

7.4.1 Shear Strength of Soil Samples

Figure 7.9 shows the change of undrained shear strength, \( s_u \) over time for soil sample B1. The swelling effect was most significant within the first 24 hours, after which the change in \( s_u \) profiles became relatively gentle. There was no noticeable change in \( s_u \) profiles from day 3 to day 7. This is very similar to the observations reported by Vlahos...
(2004) for a soil sample made from the same type of kaolin powder. The soil samples were therefore left to swell for three days before the jack-up reinstallation tests commenced.

![Graph](image-url)

**Figure 7.9: Changes of Undrained Shear Strength Profiles with Time (Sample B1)**

Figure 7.10 shows the $s_u$ profiles of the four sample boxes measured by a t-bar penetrometer 7 days after removing the soil samples from the consolidation press. The $s_u$ profiles appear to be consistent. The $s_u$ profiles also indicated that sample B2 is the strongest while B3 is the weakest soil sample. The $s_u$ profiles variation increased with increasing depth due to local variation in the soil samples. At footprint toe level ($z_F=0.02m$), the $s_u$ was within a 10% difference. Therefore, the jack-up reinstallation test results obtained from different soil samples should be comparable. The $s_u$ profiles also agreed with the empirical relationship of Ladd et al. (1977) presented in Section 3.5.2. The $s_u$ profiles may be approximated by the following bi-linear expression:

\[
\begin{align*}
    s_u &= 1.80 + 1.51z, \text{ for } z < 0.035m \\
    s_u &= 5.69 + 0.40z, \text{ for } z > 0.035m
\end{align*}
\]
7.4.2 Jack-up Installation Tests and Rigid Jack-up Reinstallation Tests

‘Rigid reinstallation test’ refers to the reinstallation of a model jack-up leg rigidly connected to the actuator, simulating a model jack-up with no flexibility at CP. The model jack-up leg was installed at an offset of 1.0D from the idealised footprint cavity type TB. The results provided baselines for comparison with results from the flexible jack-up and are therefore discussed in detail below.

Figure 7.11 presents the results of rigid reinstallation tests conducted in different soil samples. The sign convention and definition of terminologies are shown in Figure 1.8 and Figure 6.18. A positive moment at the LRP (M_{LRP}) represents soil bending the footing away from the footprint centre, and a positive horizontal force represents soil pushing the footing towards the footprint centre. The moment at the CP (M_{CP}) was extrapolated from the soil load acting at footing level (H and M_{LRP}) and represented the maximum moment acting on the model jack-up leg. According to statics theory, the horizontal force at the LRP and CP are identical and it is simply referred to as H in this study.

In the 1g-scaled model tests, the jack-up generally exhibited the same three-stage response observed in the drum centrifuge tests (refer to Section 3.5.4 for details).
Although the results at shallow depth ($z/D<0.05$) for the four rigid reinstallation tests were relatively consistent, their differences increased with penetration depth. S-R-3 and S-R-4 exhibited dual peak responses. The $H$ and $M_{CP}$ at deep reinstallation ($z/D=0.5$) both exhibited secondary peaks. This is likely to be related to the local $s_u$ variations near $z/D=0.5$ in soil samples B3 and B4. The undrained shear strength profiles of the four soil samples were within 10% difference at the footprint toe level ($z=0.02m$) and the difference increased to 25% at $z/D=1D$. The results were therefore normalised by the $s_u$ profiles (different for each soil sample) of the corresponding soil samples and are compared in Figure 7.12.

The normalised vertical forces were consistent across the four tests. The normalised vertical forces increased from touchdown level and reached their peak values at approximately $z/D=0.5$. The peak values from the four tests ranged from 8.5 to 9, and
the normalised vertical forces remained nearly constant with further penetration. The normalised \( M_{LRP} \) also demonstrated consistency and reached maximum values near the touchdown level. The difference in the maximum normalised \( M_{LRP} \) was within 30% difference. The \( H \) showed considerable variations. Although the maximum normalised \( H \) occurred at similar depths \((z/D=0.2)\), the four tests showed horizontal forces that differed by 40%. S-R-2 produced the largest horizontal force, and S-R-3 gave the lowest. The largest \( H \) (S-R-2) was from the reinstallation test conducted in the strongest soil sample. The smallest \( H \) (S-R-3) occurred in the softest soil sample. These results suggest that \( H \) is sensitive to the soil strength and that \( H \) increases with increasing soil strength. However, \( H \) started to converge from \( z/D=0.3 \). The \( M_{CP} \) was influenced by \( H \) variations and also varied significantly at shallow depths. The \((M_{CP})_{max}\) from S-R-2 was 60% larger than the \((M_{CP})_{max}\) from S-R-3.

As shown in Figure 7.11, S-R-3 and S-R-4 show secondary peaks at \( z/D=0.4 \) and relatively large variations in their horizontal forces and moments during relatively deep reinstallation \((z/D>1.0)\). This result could be related to disturbed soil being trapped beneath the footing and inherent local \( s_u \) variations in the soil sample. However, the normalised forces and moments show distinct peak values above the footprint toe level \((z/D=0.33)\). As penetration deepened, the normalised forces and moments were quite consistent and fluctuated around zero during deep reinstallation. Future discussions will therefore focus on the shallow depth response, which appears to be more critical.
In order to validate the relevance of 1g-scaled model tests results, the rigid reinstallation test results are compared with an identical test performed in the 250g gravity environment. All tests utilised a fully-fixed model leg actuator, an offset distance of 1.0D and the footprint geometry TB. The TB-2D-10D test results modelled the reinstallation response in field condition using the drum centrifuge (refer to Chapter 3 for details).

The comparison is presented in Figure 7.12. The 1g-scaled model test results exhibited less local variations compared to TB-2D-10D. The same three-stage response (see details in Section 3.5.4) was observed in both 1g-scaled model test and drum centrifuge test results. The maximum $M_{LRP}$ from both 1g-scaled model test and drum centrifuge test occurred near the touchdown level. The maximum normalised $M_{LRP}$ from S-R-1 was almost identical to the drum test results. The maximum $H$ from the 1g-scaled
model tests occurred at z/D≈0.20, shallower than TB-2D-10D (at footprint toe level z/D=0.33). The maximum normalised horizontal forces from the 1g-scaled model tests were also 50-70% smaller than the drum centrifuge test results.

As shown above, consistent, low-noise test results were obtained from the 1g-scaled model tests. The comparison also presented the potential difference between the peak loadings obtained from 1g-scaled model tests compared to the centrifuge test (representing field condition). In particular, the horizontal forces from 1g-scaled model tests were significantly lower than the centrifuge test. However, the same three-stage response could be observed from both the 1g-scaled model tests and the drum centrifuge tests. The 1g-scaled model tests provide meaningful and relevant insights into the behaviour of a jack-up reinstalled nearby existing footprints.

### 7.4.3 Summary of Experimental Results

All of the experimental results are grouped by flexibility type and presented in Section 7.4.4 to Section 7.4.7. To facilitate comparison, the maximum and the normalised horizontal force and moment values are summarised in Table 7.6. The maximum movements at the common point of the jack-up model are summarised in Table 7.7.

Table 7.6: Summary of Peak Loads from 1g- Scaled Model Tests on Jack-up Reinstallation near Idealised Footprint Cavity

<table>
<thead>
<tr>
<th>Test Name</th>
<th>$H_{\text{max}}$ (N)</th>
<th>$(H/\text{As}<em>u)</em>{\text{max}}$</th>
<th>$(M_{\text{LRP}})_{\text{max}}$ (Nm)</th>
<th>$(M_{\text{LRP}}/\text{As}<em>uD)</em>{\text{max}}$</th>
<th>$(M_{\text{CP}})_{\text{max}}$ (Nm)</th>
<th>$(M_{\text{CP}}/\text{As}<em>uD)</em>{\text{max}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-R-1</td>
<td>2.84</td>
<td>0.34</td>
<td>0.33</td>
<td>1.03</td>
<td>-0.783</td>
<td>-1.567</td>
</tr>
<tr>
<td>S-R-2</td>
<td>3.78</td>
<td>0.44</td>
<td>0.32</td>
<td>0.97</td>
<td>-0.922</td>
<td>-1.696</td>
</tr>
<tr>
<td>S-R-3</td>
<td>2.21</td>
<td>0.31</td>
<td>0.29</td>
<td>1.04</td>
<td>-0.623</td>
<td>-1.045</td>
</tr>
<tr>
<td>S-R-4</td>
<td>2.55</td>
<td>0.33</td>
<td>0.27</td>
<td>0.90</td>
<td>-0.686</td>
<td>-1.125</td>
</tr>
<tr>
<td>S-H-1</td>
<td>2.26</td>
<td>0.20</td>
<td>0.29</td>
<td>0.84</td>
<td>-0.610</td>
<td>-0.771</td>
</tr>
<tr>
<td>S-H-2</td>
<td>2.42</td>
<td>0.23</td>
<td>0.30</td>
<td>0.84</td>
<td>-0.580</td>
<td>-0.947</td>
</tr>
<tr>
<td>S-M-1</td>
<td>2.25</td>
<td>0.23</td>
<td>0.26</td>
<td>0.73</td>
<td>-0.615</td>
<td>-0.891</td>
</tr>
<tr>
<td>S-M-2</td>
<td>1.38</td>
<td>0.24</td>
<td>0.22</td>
<td>0.71</td>
<td>-0.324</td>
<td>-0.426</td>
</tr>
<tr>
<td>S-L-1</td>
<td>2.09</td>
<td>0.22</td>
<td>0.25</td>
<td>0.78</td>
<td>-0.492 (0.597)</td>
<td>-0.780 (1.536)</td>
</tr>
<tr>
<td>S-F-1</td>
<td>1.60</td>
<td>0.08</td>
<td>0.19</td>
<td>0.36</td>
<td>-0.486</td>
<td>-0.346</td>
</tr>
</tbody>
</table>
Table 7.7: Summary of Maximum Displacement from 1g- Scaled Model Tests on Jack-up Reinstallation near Idealised Footprint Cavity

<table>
<thead>
<tr>
<th>Test Name</th>
<th>hmax (mm)</th>
<th>θmax (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-H-1</td>
<td>3.30</td>
<td>-</td>
</tr>
<tr>
<td>S-H-2</td>
<td>9.51</td>
<td>-</td>
</tr>
<tr>
<td>S-M-1</td>
<td>-</td>
<td>-0.28</td>
</tr>
<tr>
<td>S-M-2</td>
<td>-</td>
<td>-0.70</td>
</tr>
<tr>
<td>S-L-1</td>
<td>3.20</td>
<td>0.20</td>
</tr>
<tr>
<td>S-F-1</td>
<td>3.76</td>
<td>-0.51</td>
</tr>
</tbody>
</table>

7.4.4 Effect of Including Horizontal Flexibility

The first phase of the 1g-scaled model tests considered the jack-up reinstallation response with a partially restrained connection at the model leg and actuator. The model jack-up had finite horizontal flexibility (C_{Hh}) and was allowed to slide according to the prescribed horizontal flexibility with the measured load. This is equivalent to a horizontal spring connection between the model jack-up leg and actuator. The results from the three tests performed with different C_{Hh} are shown in Figure 7.13.

In S-H-1 and S-H-2, the footing slid towards the footprint centre and the maximum horizontal displacements were 3.3mm (hmax/D=0.055) and 9.5mm (hmax/D=0.158) respectively. The increase in hmax was proportional to the increase in horizontal flexibility (by a ratio of 3). The Hmax decreased by 14% and 22% in S-H-1 and S-H-2, respectively, compared to the rigid reinstallation test S-R-1. The trends of change in H with penetration depth in S-H-1 and S-H-2 were similar and exhibited only an 8% difference in Hmax values. The M_{LRP} was insensitive to changes in horizontal flexibility. The values of (M_{LRP})_{max} obtained for the three tests only showed a 5% difference. Like Hmax and (M_{LRP})_{max}, the values of (M_{CP})_{max} were almost identical for S-H-1 and S-H-2. The (M_{CP})_{max} decreased by 20% and 25% in S-H-1 and S-H-2, respectively, compared to the rigid reinstallation test S-R-1.

The hmax in S-H-2 was considered to be excessive compared to the recommendation in SNAME (2002). SNAME (2002) recommended a maximum leg inclination of 0.5% leg length which is equivalent to 0.47m for a 94m tall jack-up unit. The normalised allowable horizontal movement is h/D=0.078 for a 6m diameter footing, which was only half of hmax from S-H-2 (hmax/D=0.158). Although SNAME (2002) did not specify whether the recommendation is for jack-up preloading or working condition, the maximum movement of hmax/D=0.158 in S-H-2 was considered to be excessive for jack-up reinstallation on site (Purwana, 2011). The jack-up can suffer from structural damage with the relatively large leg movement. In practice, a relatively high rack phase
difference (RPD) will be observed and the reinstallation process will be terminated. Stomping might be conducted attempting to mitigate the problem. Therefore, the range of flexibility considered in the study was large enough and covered the jack-up reinstallation response in extreme situation.
Figure 7.13: The Reinstallation Responses of 1g-Scaled Model Jack-Up with Different Horizontal Flexibilities
The change of load-displacement responses with penetration depth is described in detail below to explain the effect of including horizontal flexibility. The directions of loadings and displacements are shown in Figure 7.14. For ease of comparison, the horizontal forces and moments are presented as follows: the symbol $\uparrow$ indicates an increase, $\approx$ indicates no significant change, and $\downarrow$ indicates a reduction when compared to rigid reinstallation at the touchdown level.

Stage 1 response: When the footing came into contact with the founding soil, the soil reaction pushed the footing towards the footprint centre, and, with the horizontal flexibility considered in tests S-H-1 and S-H-2, the footing started to move horizontally towards the footprint centre. The soil reaction was released with horizontal movements, after which $H$ dropped significantly and became slightly negative. The eccentric vertical load generated a positive $M_{LRP}$ and bent the footing away from the footprint. At the CP, the positive $M_{LRP}$ was larger than the moment due to the $H$ acting at LRP (negative), therefore, giving a positive $M_{CP}$ and attempted to bend the jack-up leg away from the footprint. Both the $M_{LRP}$ and the $M_{CP}$ were positive near the touchdown level.

Stage 2 response: Contrary to the rigid reinstallation case, the $H$ values in S-H-1 and S-H-2 did not peak at $z/D=0.2$ as penetration continued. Instead, the $H$ values peaked at approximately $z/D=0.3$, after which they remained nearly constant until penetration went below $z/D=0.6$. The horizontal displacement increased almost linearly with penetration until reaching the footprint toe level ($z/D=0.33$).

After peaking near the touchdown level, the $M_{LRP}$ decreased sharply with further penetration. As a result of the increase in $H$ and the reduction in $M_{LRP}$ with penetration
depth, the $M_{CP}$ reduced sharply and became negative for penetrations below $z/D=0.05$. This indicates that the jack-up leg was bending towards the footprint centre.

The vertical forces measured during the three tests differed for penetration depths from the touchdown level to $z/D=0.5$. It could be that the footing slid further towards the footprint centre and reduced the offset distance. The vertical force decreased with decreasing offset distance, as previously discussed in Chapter 3. The vertical forces from the three tests converged and became almost identical at $z/D=0.5$, when the footing stopped sliding.

The reinstallation response of the jack-up with finite horizontal flexibility are summarised here:

- The jack-up model slid towards the footprint centre from touchdown level until reaching $z_f$;
- $h_{max}$ increased linearly with horizontal flexibility;
- When the horizontal flexibility was increased from zero (S-R-1) to finite (in test S-H-2 and S-H-1), the results demonstrated a 20% reduction in $H_{max}$ and a 5% reduction in the $(M_{LRP})_{max}$;
- When the horizontal flexibility was increased by three times from S-H-1 to S-H-2, there was no significant change in $H_{max}$ and $(M_{LRP})_{max}$.

Therefore, for a jack-up flexible in horizontal direction only, it could encounter excessive horizontal movements at the footing level, with a relatively small reduction in the leg forces during reinstallation.

### 7.4.5 Effect of Including Rotational Flexibility

Three jack-up reinstallation tests were conducted to investigate the effect of including rotational flexibility in isolation. Finite rotational flexibility was prescribed so that the model leg could rotate about CP of the jack-up model. This is equivalent to a hinge connection. Figure 7.15 shows the effects of including rotational flexibility.
Figure 7.15: The Reinstallation Responses of 1g-Scaled Model Jack-Up with Different Rotational Flexibility
When rotational flexibility was included the footing rotated towards the footprint centre. The maximum angle of rotation ($\theta_{\text{max}}$) in S-M-1 and S-M-2 was 0.275° and 0.699° respectively. $H_{\text{max}}$ and $(M_{\text{LRP}})_{\text{max}}$ were reduced by 40% and 20%, respectively, when the rotational flexibility was increased from zero to $C_{M0} = 0.532°/Nm$ (S-R-2 to S-M-1). The rotational flexibility of S-M-2 ($C_{M0} = 4.620°/Nm$) was 8.5 times larger than S-M-1. $H_{\text{max}}$ and $(M_{\text{LRP}})_{\text{max}}$ were reduced by approximately 65% when compared to S-R-2. Unlike the horizontal flexibility tests, the load-displacement relations were highly non-linear, and the $\theta_{\text{max}}$ increased by 2.5 times with an 8.5 times increase in $C_{M0}$.

Rotation at CP also induced an equivalent horizontal displacement at the LRP ($h_0$) and this concept is illustrated in Figure 7.16. The change of $h_0$ with penetration depth is shown in Figure 7.17. The length of the model jack-up leg was 235mm; therefore, the footing moved towards the footing centre by $h_0/D = 0.02$ and $h_0/D = 0.05$ for S-M-1 and S-M-2, respectively.

![Figure 7.16: Definition of Displacement](image-url)
Figure 7.17: Equivalent Horizontal Displacement Due to Rotation at CP

The directions of loading and displacements at touchdown level are shown in Figure 7.18. The change of load-displacement response with penetration depth is described in detail below to explain the effect of including rotational flexibility.

Figure 7.18: Comparisons of Load-Displacement Responses at Touchdown Level (Rigid Reinstallation and Reinstallation with Rotational Flexibility)

Stage 1 response: At touchdown level (z/D<0.05), the soil reaction pushed the footing towards the footprint centre and simultaneously bent the footing away from the footprint. At the CP, the negative moment induced by the horizontal load was larger than the positive $M_{LRP}$. Thus, the $M_{CP}$ was negative, and the footing rotated towards the
footprint centre. The soil load was then released with the rotation. The $M_{CP}$ and $M_{L,RP}$ were therefore reduced. The $H$ was also reduced as the footing rotated into the footprint. This was because there was equivalent horizontal displacement at the footing level as the jack-up leg rotated. Nonetheless, the release of the soil reaction was insufficient to keep the soil from bending the footing towards the footprint centre, and the $M_{CP}$ remained negative.

**Stage 2 response:** As penetration continued, the trends in $H$ and $M_{CP}$ variations for S-M-1 and S-M-2 began to significantly differ. In S-M-1, the footing rotated from touchdown level such that the angle of rotation increased in a linear manner until reaching maximum $M_{CP}$ near footprint toe level ($z/D \leq 0.33$). The $M_{CP}$ decreased sharply with further penetration.

The three-stage response observed from rigid reinstallation tests was not observed in S-M-2. The $H$ increased sharply at relatively shallow depths ($z/D=0.05$), then remained almost constant with further penetration. The $M_{CP}$ exhibited a similar response but with gently increasing gradient. This occurred because the jack-up leg was installed in the soil at a relatively large angle (compared to S-M-1). Thus, it experienced $H$ from the unbalanced lateral soil load on the two sides of the footing and from the uneven soil bearing resistance throughout penetration. This horizontal force, together with the vertical force acting at the level arm of $h_T$, induced a negative $M_{CP}$ throughout penetration. This concept is illustrated in Figure 7.19. As a result, the jack-up leg continued to rotate, reaching a maximum rotation angle at termination depth ($z/D=0.95$). If the angle of rotation could be adjusted back to zero in practice (for instance, by adjusting the RPD), the $H$ and the $M_{CP}$ would be expected to return to zero.

The vertical forces from the three tests were almost identical at penetration depths above the footprint toe ($z/D<0.33$). This indicates that the reduction of offset distance (from $h_0$) was not large enough to degrade the vertical resistance at shallow depth. As penetration continued, the footing rotated further into the footprint centre, causing the difference between the vertical forces to increase. As a result, the vertical force was lower than that obtained during the rigid reinstallation test. Furthermore, as the footing was installed into the soil at an angle, the lower projected area and the sloping base of the footing also gave a lower vertical resistance (Figure 7.19). Test S-M-2 showed the most significant reduction because the rotation angle was larger than that of S-M-1.
The reinstallation response of jack-up with finite rotational flexibility is summarised here:

- The jack-up leg rotated towards the footprint centre and the maximum angle of rotation ($\theta_{\text{max}}$) in S-M-1 and S-M-2 was 0.275° and 0.699° respectively;
- The three-stage response was not observed when increasing the rotational flexibility from zero in S-R-2 to finite in S-M-2;
- The $H_{\text{max}}$ and the $(M_{\text{LRP}})_{\text{max}}$ were reduced by 65% when increasing the rotational flexibility from zero in S-R-2 to finite in S-M-2.

For a jack-up with sufficiently large rotational flexibility, the three-stage reinstallation response was no longer observed and a relatively large reduction in $H_{\text{max}}$ and $(M_{\text{LRP}})_{\text{max}}$ was observed when compared to rigid reinstallation.

### 7.4.6 Effect of Including Linearly Combined Flexibilities

The linearly combined horizontal and rotational flexibilities was adopted in phase two. S-L-1 modelled an idealised jack-up with a combined horizontal spring and hinge connection between the model jack-up leg and actuator. The results are shown in Figure 7.20. With the linearly combined flexibilities established at the CP, the jack-up
leg slid towards and rotated away from the footprint centre at the same time. The $h_{\text{max}}$ was 3.20mm ($h_{\text{max}}/D=0.05$) and the $\theta_{\text{max}}$ was approximately 0.2°. Because the rotational and the horizontal movements were in opposite directions, the horizontal displacement at the footing level was combined as in Figure 7.21. The total horizontal movement, $h_{\text{total}}$ was 2.38mm ($h_{\text{total}}/D=0.04$) towards the footprint centre. The magnitudes of $H_{\text{max}}$ and $(M_{\text{CP}})_{\text{max}}$ were similar to those for the rigid reinstallation case, but $(M_{\text{LRP}})_{\text{max}}$ was reduced by 13% compared to the rigid reinstallation scenario.
Figure 7.20: The Reinstallation Response of 1g-Scaled Model Jack-Up with Linearly Combined Horizontal-Rotational Flexibilities
Figure 7.21: Total Horizontal Displacements of S-L-1

Figure 7.22 shows the direction of loadings and displacements of the reinstalling leg. The detailed description on the change of load-displacement responses with penetration depth is presented below to explain the effect of including linearly combined horizontal and rotational flexibility.

**Stage 1 response:** The footing started to slide towards the footprint centre as the footing came into contact with the soil. The H then decreased sharply in response to the horizontal movement, while there was no significant change in the M_{LRP}. As a result of the negative H and the positive M_{LRP}, the M_{CP} increased sharply in an attempt to bend the footing away from the footprint centre. As a result, the footing rotated in the same direction.
Stage 2 response: As the $H$ increased with further penetration, the $M_{CP}$ and the rotation angle were both reduced. The $\text{H}_{\text{max}}$ and the $\left(M_{CP}\right)_{\text{max}}$ occurred at $z/D=0.2$ and $z/D=0.1$, respectively. The $H$ remained almost constant until penetration beyond $z/D=0.5$, at which point the $H$ began to decrease.

The vertical force of S-L-1 was lower than that of the rigid reinstallation test throughout penetration. It is because the jack-up leg was installed in the soil at an angle, further penetration resulted in reduced vertical resistance.

The reinstallation response of jack-up with linearly combined horizontal and rotational flexibility are summarised here:

- The jack-up model slid towards and rotated away from the footprint centre at the same time;
- Including the linearly combined flexibility has very limited effect on the peak response during reinstallation. The $(M_{LRP})_{\text{max}}$ was only reduced by 13% and there was no change in $\text{H}_{\text{max}}$ and $(M_{CP})_{\text{max}}$ compared to the rigid reinstallation scenario.

The load-displacement responses during reinstallation were different when compared to the case of the isolated horizontal/rotational flexibility. It is due to the complex interaction between the release in horizontal and moment load with the combined flexibility. However, changing the connection flexibility from rigid to linearly combined flexibility had very limited effect on the peak loadings.

### 7.4.7 Effect of Including Cross-Coupled Flexibilities

The full jack-up model presented in Figure 7.2 is considered in the third phase of investigation. With the cross-coupled horizontal and rotational flexibilities prescribed at the CP, the reinstallation response of the jack-up could be realistically modelled. The results are shown in Figure 7.23. The footing slid and rotated towards the footprint centre and the peak loading also reduced significantly. The $h_{\text{max}}$ was 3.76mm ($h_{\text{max}}/D=0.063$) and $\theta_{\text{max}}$ was 0.51°. Because both the sliding and rotation were in the same direction, the maximum total displacement at the LRP was approximately 5.8mm ($h_{\text{total}}/D=0.097$). The $(M_{LRP})_{\text{max}}$ was reduced by 30% compared to the rigid reinstallation test. The $\text{H}_{\text{max}}$ and $(M_{CP})_{\text{max}}$ from S-F-1 were approximately 65% of those from S-R-4.
Figure 7.23: The Reinstallation Response of 1g-Scaled Model Jack-Up with Cross-Coupled Horizontal and Rotational Flexibilities
The contributions of H and M\textsubscript{CP} to the jack-up movement were decoupled according to Figure 7.24. Figure 7.25 shows the decoupled horizontal displacement and angular movement. The H contributed to 65% of horizontal movement and 53% of angular movement.

\[
\Delta h = C_{Hh} \Delta H + C_{Mh} \Delta M_{cp}
\]
\[
\Delta \theta = C_{H\theta} \Delta H + C_{M\theta} \Delta M_{cp}
\]

\[
\begin{bmatrix}
\Delta h \\
\Delta \theta
\end{bmatrix} =
\begin{bmatrix}
C_{Hh} & 0 & C_{Mh} \\
0 & 0 & 0
\end{bmatrix}
\begin{bmatrix}
\Delta H_{cp} \\
\Delta M_{cp}
\end{bmatrix} +
\begin{bmatrix}
\Delta V_{step}
\end{bmatrix}
\]

Note: \( \Delta H_{cp} = \Delta H \)

**Figure 7.24: Decoupling of Movement**

The directions of loading and displacements of the reinstalling jack-up leg at touchdown level are shown in Figure 7.26. At touchdown level, the soil was imposing a positive H on the footing. This H resulted in a positive horizontal displacement and a negative rotation (\( C_{H\theta} \) being negative) around the CP. The footing therefore slid towards and rotated into the footprint at the same time. This soil load at the footing was released with the movement, and both the H and the M\textsubscript{LRP} were reduced to almost zero. Therefore, instead of having a positive M\textsubscript{CP}, as in S-L-1, a negative M\textsubscript{CP} was observed. The negative M\textsubscript{CP} generated negative angular movements and positive horizontal movement (\( C_{Mh} \) being negative).

**Figure 7.25: Decoupled Horizontal Displacement and Angular Movement**
Chapter 7: Combined Effect of Footprint Geometry and Jack-up Structural Properties on Reinstallation

The three-stage response observed during rigid reinstallation tests was not observed in S-F-1. As the footing slid towards and rotated into the idealised footprint cavity, the H and MCP only increased gradually with further penetration. The H and \( M_{LRP} \) increased gently below \( z/D=0.5 \). A similar trend was observed for the MCP. The constant H and the MCP were caused by the inclination of the jack-up leg. As a result the footing continued to move until the penetration depth was near \( z/D=0.8 \). Due to the inclined footing penetration with lower projected area, the vertical force acting on the footing was lower than rigid reinstallation near the touchdown level. As a result the \( (M_{LRP})_{max} \) was only 60% of S-R-4.

The reinstallation response of a full jack-up model are summarised here:

- The reinstallation response did not exhibit a three-stage response, instead, after a rapid increase near touchdown level, the loading at the CP then continued to increase gently until near \( z/D=0.8 \);
- The jack-up leg slid and rotated towards the footprint centre with a significant reduction in the horizontal force and moment.

7.5 CONCLUDING REMARKS

The effect of structural properties to the jack-up reinstallation response was investigated by real-time hybrid testing of a scaled jack-up model at unit gravity. The 1g-scaled model tests provided low-noise and consistent results for the exploration of the complex load-displacement response during jack-up reinstallation. A wide range of flexibilities
were adopted to cover the extreme reinstallation scenario. The key findings are summarized below:

- In phase one test, when the connection between the jack-up leg and actuator changed from fully fixed to partially constrained in the horizontal direction, there was no significant change in the peak loadings;

- When the jack-up leg is partially constrained in horizontal direction, the footing slid towards the footprint centre. The $h_{\text{max}}$ from the two horizontal flexibility tests increased linearly with the horizontal flexibility;

- When the connection was partially constrained in the angular direction, the peak loading reduced significantly and the footing rotated towards the footprint centre;

- With a relatively large rotational flexibility ($C_{M0}=4.62^\circ/\text{MNm}$) prescribed at the CP, the three-stage response was no longer observed;

- When a linearly combined horizontal and rotational flexibility was adopted, the footing slid and rotated away from the footprint centre at the same time. However, the $(M_{\text{LRP}})_{\text{max}}$ only reduced by 13% without changes in $H_{\text{max}}$ when comparing to rigid reinstallation;

- A full jack-up model was considered with the implementation of cross-coupled horizontal-rotational flexibility at the CP, the jack-up leg slid and rotated towards the footprint centre at the same time. The maximum movement was $h_{\text{max}}/D=0.063$ and $\theta_{\text{max}}=0.51^\circ$. Both the $H_{\text{max}}$ and $(M_{\text{LRP}})_{\text{max}}$ reduced by approximately 60% when compared to rigid reinstallation.

- The three-stage response was not observed during the reinstallation of the full jack-up model, instead, the $H$ and $M_{\text{LRP}}$ were increased rapidly at touchdown level, followed by a gentle increased with penetration depth.

In this chapter, reinstallation response near idealised footprint cavities was considered which provided valuable insight to the shallow depth reinstallation response. The results demonstrated it is important to include the effect of structural properties in the modelling of jack-up reinstallation response. Although the peak loadings reduced significantly when comparing to rigid reinstallation, the jack-up moved towards the footprint centre and could potentially collide with adjacent fixed platform. Real footprints are considered in the next chapter for the investigation of the combined effect of footprint geometry, soil heterogeneity and jack-up structural properties to the reinstallation response.
CHAPTER 8. PHYSICAL MODELLING OF THE COMPLETE JACK-UP REINSTALLATION PROBLEM

8.1 INTRODUCTION

The real-time hybrid testing method has been successfully applied to the investigation of reinstallation response near an idealised footprint cavity, as discussed in Chapter 7. In this chapter, the investigation was extended to consider the reinstallation response near a real footprint. The beam centrifuge facility at UWA was used to create real footprints with variable soil properties by penetrating and extracting jack-up model. Jack-up units with finite structural flexibilities were then reinstalled nearby the real footprints. This allowed the investigation of reinstallation response with a complete model considering all three governing parameters: footprint geometry, footprint soil properties and jack-up structural properties (Figure 8.1).

![Figure 8.1: The Three Parameters Modelled by Real-time Hybrid Testing Method in the Beam Centrifuge](image)

8.2 JACK-UP MODEL

The dimensions and properties of the jack-up unit are presented in Figure 7.2. However, the foundation stiffness values presented in Table 7.2 are modified here. This is because the undrained shear strength, $s_u$, of the soil samples adopted in the beam centrifuge was different to the strength adopted in the scaled model. Consequently, a different set of foundation stiffness values were evaluated as shown in Table 7.2. To capture a wide range of jack-up movements, SNAME foundation stiffness ($F_{\text{SNAME}}$) and
a lower bound assuming one-tenth of the SNAME foundation stiffness \((F_{0.1xSNAME})\) were considered. The associated flexibility matrices are presented in Table 8.2.

### Table 8.1: Soil Properties and Foundation Stiffness Values \((F_{SNAME})\)

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Beam Centrifuge Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>(D)</td>
<td>6m</td>
</tr>
<tr>
<td>(kD/sum)</td>
<td>1.28</td>
</tr>
<tr>
<td>(s_u) at (z/D=0.25)</td>
<td>4.70 kPa</td>
</tr>
<tr>
<td>(OCR) at (z/D=0.25)</td>
<td>4.80</td>
</tr>
<tr>
<td>(K_v)</td>
<td>52583.31 kN/m</td>
</tr>
<tr>
<td>(K_h)</td>
<td>40542.49 kN/m</td>
</tr>
<tr>
<td>(K_{rot})</td>
<td>351165.03 kNm/rad</td>
</tr>
</tbody>
</table>

### Table 8.2: Flexibility Matrices of Jack-up Model for the Beam Centrifuge Tests at 100g

<table>
<thead>
<tr>
<th>Flexibility Matrices for Centrifuge Test (mm/N, mm/Nm, deg/N or deg/Nm)</th>
<th>Prototype Dimension</th>
<th>Model Dimension (@100g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-FSNAME</td>
<td>1.57E-02 0 -2.60E-04</td>
<td>1.57E+00 0 -2.60E+00</td>
</tr>
<tr>
<td>(-3.13E-03) 0 -2.68E-04</td>
<td>(-3.13E-01) 0 -2.68E+00</td>
<td></td>
</tr>
<tr>
<td>(-1.49E-05) 0 6.14E-07</td>
<td>(-1.49E-01) 0 6.14E+00</td>
<td></td>
</tr>
<tr>
<td>C-F0.1xSNAME</td>
<td>2.28E-02 0 4.13E-05</td>
<td>2.28E+00 0 4.13E+01</td>
</tr>
<tr>
<td>1.25E-02 0 -9.34E-04</td>
<td>1.25E+00 0 -9.34E+00</td>
<td></td>
</tr>
<tr>
<td>2.37E-06 0 1.35E-06</td>
<td>2.37E+00 0 1.35E+00</td>
<td></td>
</tr>
</tbody>
</table>

In addition to the jack-up model shown in Figure 7.2 (namely S1), a short jack-up unit (namely S2), was also considered in this study. The dimensions and properties of the short jack-up model are identical to the original jack-up model except that the leg length was shortened from 94m to 23.5m. This simulated the same jack-up unit operating in shallower water depths. Figure 8.2 shows the numerical model of the short jack-up unit. The flexibility matrix describing the load-displacement response of the short jack-up unit with lower bound foundation stiffness \((F_{0.1xSNAME})\) is shown in Table 8.3.
The full flexibility matrix was modified according to the three testing phases presented in Figure 7.3. In addition, a forth testing phase was established to model the jack-up unit with vertical-horizontal-moment cross-coupled flexibilities at the common point, CP. The vertical coupling terms allow the jack-up leg to rise/penetrate according to the horizontal and moment load combination. The modified full flexibility matrix is shown in Figure 8.3.
8.3 EXPERIMENTAL APPARATUS, PROCEDURES AND PROGRAMME

The aim of the experimental programme was to investigate the combined effect of the heterogeneous soil strength within the footprint, the footprint geometry and the jack-up structural properties using real-time hybrid testing in the beam centrifuge. The real footprints were created by penetration and extraction of the footing into the soil under high gravity environment in the beam centrifuge, followed by jack-up reinstallation at an offset distance of 1.0D.

8.3.1 Beam Centrifuge Facility at UWA

Established in 1989, the beam centrifuge at UWA has a maximum acceleration of 200g (Figure 8.4). The centrifuge has a swinging platform at a radius of 1.8m from the main axis. A strongbox containing soil samples and associated equipment can be mounted on the platform. The strongbox is balanced by a counter-weight at the other end of the cross-beam. As the centrifuge spins, the cradle freely swings up, and the increasing acceleration acts perpendicularly to the base of the platform. Figure 8.5 shows the strongbox and the VHM actuator arrangement in the beam centrifuge.

The beam centrifuge is housed in an air-conditioned, circular reinforced concrete chamber. This ensures that a constant temperature is maintained throughout the test, thereby avoiding temperature variations that might affect the soil sample and the sensitive instrumentation. The flight computer is mounted on the low-g central platform of the centrifuge to perform the various functions outlined in Section 6.3.1. There is a control room housing two computers for remote control and monitoring of centrifuge operation. Communication with the centrifuge is via optical fibre cables and slip-rings, which are housed within the main axis of the centrifuge. The centrifuge and associated equipment are described in detail by Randolph et al. (1991).
8.3.2 Preparation of Soil Sample and Testing Procedures

The kaolin slurry was prepared according to Section 4.4.1 and was consolidated with a maximum consolidation pressure of 47kPa. The soil sample was then consolidated at 120g in the beam centrifuge for two days. At final height, the soil sample was approximately 210mm. The beam centrifuge was then ramped down to 100g for testing. The undrained shear strength profile of the slightly over-consolidated clay sample was determined by t-bar penetrometer tests prior to any testing.
The real footprint was created by penetration and extraction of the physical jack-up model into the soil. The model jack-up unit was then reinstalled at 1.0D from the real footprint. Table 8.4 shows the testing procedures in detail.

**Table 8.4: Testing Procedures of Real-time Hybrid Testing in Beam Centrifuge**

<table>
<thead>
<tr>
<th>Setting up of the Test</th>
<th>Footprint Creation</th>
<th>Jack-up Reinstallation</th>
<th>Setting up for Next Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>1) Mount the strongbox to the swinging platform of the beam centrifuge;</td>
<td>6) Start real-time hybrid testing control programme;</td>
<td>11) Move model jack-up leg to the reinstallation site at 1.0D away from footprint centre;</td>
<td>16) Ramp down the centrifuge.</td>
</tr>
<tr>
<td>2) Fix the testing equipments: VHM actuator, model jack-up leg, light and camera to the strongbox;</td>
<td>7) Load zero flexibility matrix for purely vertical penetration of model jack-up leg;</td>
<td>12) Wait for 20 mins after complete extraction of model jack-up leg;</td>
<td>17) Clean the model jack-up leg and remove the soil lump on footing;</td>
</tr>
<tr>
<td>3) Start the data logging system;</td>
<td>8) Install the model jack-up leg to 70mm below soil surface at 0.1mm/s;</td>
<td>13) Load ( C_{\text{loading}} ) and ( C_{\text{unloading}} ) of testing jack-up unit;</td>
<td>18) Move the testing equipment to another test site;</td>
</tr>
<tr>
<td>4) Spin up the beam centrifuge to 100g level;</td>
<td>9) Hold the model jack-up leg at the maximum penetration depth (70mm below soil surface) for 10 mins;</td>
<td>14) Install the model jack-up leg to 100mm below soil surface at 0.1mm/s;</td>
<td>19) Repeat step 3-15 for another test.</td>
</tr>
</tbody>
</table>
8.3.3 Footprint Creation

The soil strength within a footprint is highly variable and is related to the strength of the undisturbed soil, the soil stress history, the depth of penetration, the preloading vertical load level, the offset distance, the operation time (OT) and the elapsed time (ET) between jack-up visits. To ensure similarity between tests the footprint creation and the reinstallation procedures were identical for all tests.

The soil samples were carefully prepared so that the $s_u$ profiles were similar. The same maximum penetration depth during footprint creation and the same preloading magnitude were used in the creation of real footprints. The soil sample undergoes re-consolidation during the jack-up operation period and after the extraction of the footing. Time is, therefore, a critical variable. The typical OT of a jack-up unit at a site ranges from a week up to two years. The ET between jack-up unit visits varies greatly for different projects. A jack-up unit might return quickly to a site for production enhancement of an oil gas platform. In some instances, a jack-up unit may not require a revisit for a period of over 5 years.

In this study, the footprint was first created by penetrating the physical jack-up model to 70mm ($z_{FC}/D=1.16$) into the soil sample, and the model was held at this depth for 10 mins. The holding time simulated an OT of 70 days on the prototype scale. The jack-up was then reinstalled 20 mins after the complete extraction of the physical jack-up model. This simulated an ET of 140 days. The reinstallation was conducted to a maximum depth of 100mm ($z_{max}$), which was 0.5D below $z_{FC}$. This allowed the determination of the influential depth of the real footprint (Figure 8.6). However, the maximum bearing resistance during footprint creation was only around 75kPa. This is lower than the operation range of a typical jack-up, which is between 200 to 600kPa (Osborne et al. 2006). Although higher bearing resistance could be achieved with deeper penetration, the jack-up leg would likely be influenced by the boundary effect as it approached the bottom of the strongbox.

Only one offset distance, $\beta=1.0D$, was considered in order to maintain the study’s focus on the structural properties of jack-up unit. Figure 8.6 illustrates the process and defines the terminologies.
To further assess the time effect of reinstatement procedures to the footprint properties, the percentage of excess pore pressure dissipation was estimated based on Gan (2009). Gan (2009)’s chart on dissipation of excess pore pressure dissipation versus time are shown in Figure 8.7 and Figure 8.8. Charts for both normally-consolidated and over-consolidated soils of the same kaolin clay used in the experiments are presented because the response of the slightly over-consolidated soil sample adopted in this study was expected to lie between the two. The modified equation for time factor recommended by Gan (2009) is also presented:

\[
\tau = \frac{4c_v t}{D^2} \tag{8.1}
\]

The time factor, \( \tau \) is proportional to time, \( t \), the coefficient of consolidation, \( c_v \) and inverse of footing diameter, \( D \). For the 10 mins OT and 20 mins ET, \( \tau \) is 0.007 and 0.0135, respectively.

As shown in Figure 8.7 and Figure 8.8, a maximum of 10% of excess pore pressure dissipation occurred during the operation time and elapsed time considered in this study. As the soil consolidated with dissipation of excess pore pressure, the soil around the footprint could regained some of its strength.
8.3.4 List of Beam Centrifuge Tests and As-built Layout of Test Sites

A total of six tests were conducted in two different soil samples. A summary is presented in Table 8.5. The nomenclature of the tests is presented in Section 7.3.4. In each soil sample, a rigid reinstalltion test employing zero flexibility at the model leg and actuator connection (R) was conducted as a control test. The horizontal flexibility and rotational flexibility were first investigated separately, then a phase 4 test, that is
jack-up reinstallation test with a full flexibility matrix at CP, was considered. Figure 8.9 shows a test site after the completion of the test.

Table 8.5: List of Beam Centrifuge Tests

<table>
<thead>
<tr>
<th>Test Name</th>
<th>Test Phase</th>
<th>H Flexibility</th>
<th>M Flexibility</th>
<th>Modified Flexibility Matrix</th>
<th>Foundation Stiffness</th>
<th>Sample Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-R-1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>( c = \begin{pmatrix} 0 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 0 \end{pmatrix} )</td>
<td>Rigid</td>
<td>BB1</td>
</tr>
<tr>
<td>B-R-2</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
<td></td>
<td>BB2</td>
</tr>
<tr>
<td>B-H-1</td>
<td>1</td>
<td>✓</td>
<td>-</td>
<td>( c = \begin{pmatrix} 0.815 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 0 \end{pmatrix} )</td>
<td>F_{01xSNAME}</td>
<td>BB1</td>
</tr>
<tr>
<td>B-M-1</td>
<td>1</td>
<td>-</td>
<td>✓</td>
<td>( c = \begin{pmatrix} 0 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 0.614 \end{pmatrix} )</td>
<td>F_{SNAME}</td>
<td>BB2</td>
</tr>
<tr>
<td>B-M-2</td>
<td>1</td>
<td>-</td>
<td>✓</td>
<td>( c = \begin{pmatrix} 0 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 0 \ 0 &amp; 0 &amp; 1.350 \end{pmatrix} )</td>
<td>F_{01xSNAME}</td>
<td></td>
</tr>
<tr>
<td>B-F-1</td>
<td>4</td>
<td>✓</td>
<td>✓</td>
<td>( c = \begin{pmatrix} 2.280 &amp; 0 &amp; -0.413 \ 1.250 &amp; 0 &amp; 9.340 \ -0.024 &amp; 0 &amp; 1.350 \end{pmatrix} )</td>
<td>F_{01xSNAME}</td>
<td>BB2</td>
</tr>
</tbody>
</table>

Figure 8.9: Soil Sample after Completion of the Beam Centrifuge Tests
8.4 BEAM CENTRIFUGE MODELLING - EXPERIMENTAL RESULTS

8.4.1 Shear Strength of Soil Samples

Figure 8.10 shows the $s_u$ profiles obtained from t-bar characterisation tests in the soil samples used in the beam centrifuge tests. The $s_u$ profiles namely, BB1-T1 and BB2-T1 show good consistency between the two boxes. The local peaks observed at $z/D$=0.58 and 1.33 were likely due to the soft layer formed by the topping up sequence during the soil sample preparation. These $s_u$ profiles are in agreement with the empirical relationship of Ladd et al. (1977) and fit the bi-linear approximation of

$$s_u = 2.00 + 0.65z, \text{ for } z \leq 4.5m$$  \hspace{1cm} (8-2)

$$s_u = 0.43 + 1.00z, \text{ for } z > 4.5m$$  \hspace{1cm} (8-3)

B1-T7, which is an example $s_u$ profiles from the scaled model test soil sample, is also shown in Figure 8.10. Although the soil samples adopted in the beam centrifuge tests and in the scaled model tests were prepared with different consolidation pressures, the $s_u$ profiles within the critical zone from the touchdown level to the footprint toe level ($0<z/D \leq 0.33$) from the two set of tests were in agreement. Therefore, the jack-up reinstallation test results conducted in different beam centrifuge test samples and the scaled model test soil samples are comparable. Below the footprint toe level, B1-T7 indicates that the soil sample used in the scaled model test has a higher $s_u$ than in the beam centrifuge tests. It is because at a certain range of depth ratio ($z/D$), the combination of larger over-consolidation ratio (OCR) and smaller vertical stress of soil sample B1 gave it a higher $s_u$ for soil samples used in scaled model tests than in the beam centrifuge tests.
8.4.2 Jack-up Installation Test and Typical Jack-Up Reinstallation Test

The results of jack-up during first installation (footprint creation) are firstly presented to provide baselines for comparison with jack-up reinstallation test results. Figure 8.11 shows the vertical, horizontal and moment responses of the jack-up leg during first installation (footprint creation) and subsequent rigid reinstallation test. The maximum depth of footprint creation $z_{FC}$ is also shown. B-R-1-In refers to the responses during first installation. It was found that a slight non-zero horizontal force and moment acted on the jack-up leg throughout the first installation. This is likely to be due to the non-perfect soil-footing contact near touchdown level and the slight local variation in soil strength of the soil sample. Slight inclination of the jack-up leg could also be a reason.
In test B-R-1, the jack-up leg was then installed at an offset of 1.0D from the footprint created in B-R-1-In. The jack-up leg was rigidly connected to the actuator connection and is referred to as rigid reinstallation from now on. The response of B-R-1 can be associated to a three-stage response. This three-stage response is highly influenced by the $s_u$ profile near $z_{FC}$. The stages are as follows: 1) from touchdown to footprint toe level ($0<z<0.33D$), 2) between footprint toe level and maximum penetration depth during footprint creation ($0.33D<z=z_{FC}$) and 3) below maximum penetration depth during footprint creation ($z>z_{FC}$).

The vertical force during rigid reinstallation was initially lower than the first installation, and this difference was reduced with further penetration. For over-consolidated soil samples, the $s_u$ near $z_{FC}$ could be even higher than the undisturbed soil (Gan 2009). It is because the disturbed soil regained strength as excess pore pressure dissipated during
elapsed time. Therefore, the vertical responses of initial penetration and reinstallation were comparable when the footing approached \( z_{FC} \).

The horizontal force, \( H \), increased from touchdown level, pushing the footing towards the footprint centre. After reaching its positive peak value, \( H_{\text{max}} \), at footprint toe level \( (z_t=0.33D) \), the \( H \) decreased and became negative with deeper penetration, reaching a negative peak value at \( z_{FC} \). The \( H \) at shallow depths was due to the horizontal movement of soil block and unbalanced lateral earth pressure. As the footing came close to \( z_{FC} \), the stronger soil near the footprint centre pushed the footing away from the footprint centre, giving a negative \( H_{\text{max}} \).

The moment at the LRP, \( M_{\text{LRP}} \), reached its peak value, \( (M_{\text{LRP}})_{\text{max}} \), near touchdown level. \( M_{\text{LRP}} \) then remained almost constant until reaching \( z/D=0.7 \) before decreasing to a negative peak value at \( z>z_{FC} \). At shallow depths, the combined influence of footprint geometry and uneven bearing resistance bent the jack-up leg away from the footprint centre. Figure 8.12 and Figure 8.13 show that as penetration increased, the soil strength variation under the footing changed and the soil near the footprint centre has higher strength. The stronger soil bent the footing in the reverse direction and gave a negative \( (M_{\text{LRP}})_{\text{max}} \). The negative \( H_{\text{max}} \) and \( (M_{\text{LRP}})_{\text{max}} \) began to decrease at a penetration below 1.3D and 1.5D, respectively.

The shallow depth peak horizontal force, \( H_{\text{max, s}} \), and deep reinstallation peak horizontal force, \( H_{\text{max, d}} \), were 0.04MN and -0.09MN respectively. The deep reinstallation peak moment at LRP, \( (M_{\text{LRP}})_{\text{max, d}} \) is more than three times larger than the shallow depth value \( (M_{\text{LRP}})_{\text{max, d}} \). This implies that the response at deep reinstallation is more critical than that at shallow depths. The soil initially pushed the footing towards the footprint centre, similar to what happened when reinstallation occurred near the idealised footprint cavity. When the footing came close to the stronger soil near \( z_{FC} \), the loading reversed in direction. Figure 8.14 illustrates the change of soil loading and footing responses during reinstallation.
Figure 8.12: $s_u$ Contour Map of Real Footprint in Normally-Consolidated Clay (after Gan 2009)

Figure 8.13: $s_u$ Contour Map of Real Footprint in Over-Consolidated Clay (after Gan 2009)
8.4.3 Comparison with Gan (2009)

The experimental results from Gan (2009), which also considered jack-up reinstallations near real footprints, are shown in Figure 8.15 and Figure 8.16 for comparison. Although a deeper $z_{FC}$ and shorter operation time (2.5 months) and elapsed time (0.5 year) were adopted in this study, the response trends of the two studies are comparable. Similar to the results presented in Gan (2009) for over-consolidated soil, B-R-1 exhibited dual peaks in the horizontal force and moment responses. One peak response occurred near the footprint toe level ($z/D=0.33$) and another near $z_{FC}$. In addition, in this study the peak loading reinstallation during deep penetration, $H_{max,d}$ and $M_{max,d}$, were larger than the shallow depth one, similar to the results that Gan (2009) obtained in normally-consolidated soil. In this study, which adopted a slightly over-consolidated soil, the OCR decreased with depth to a constant value of 1.2. The deep reinstallation response could thus be similar to the normally-consolidated soil, which had an OCR of 1.0. The consistency in the results validates the physical modelling technique adopted in this study. The comparison also highlights the importance of the soil strength profile of undisturbed soil to the jack-up reinstallation response.
8.4.4 Combined Effect of Footprint Geometry and Soil Properties

The combined effect of footprint geometry and soil heterogeneity was also assessed by comparing B-R-1 with the reinstallation test TB-2D-10D, which considered reinstallation near an idealised footprint cavity. The comparisons are presented in Figure 8.17. The two tests were conducted with the same connection conditions between the jack-up leg and the actuator (fully-fixed). The same offset distance ($\beta=1.0D$) and footprint geometry (TB) were adopted in the two tests. However, the $s_u$ profiles of the two soil samples were different and normalised results were therefore compared. The vertical responses obtained during the first installations of the two tests (FS and B-R-1-In) show good consistency, indicating that the comparison between the two sets of reinstallation tests is relevant.
The vertical force of the jack-up leg reinstalled near the idealised footprint cavity was higher than that obtained in the real footprint case. It is because the soil at the idealised footprint cavity was undisturbed and had higher soil strengths at shallow depths. However, as penetration continued, the difference between the two tests decreased, and they became similar at $z/D=0.33$.

The higher soil strength of the idealised footprint cavity at shallow depth also gave a higher $H$ and $M_{\text{LRP}}$ compared to the real footprint. When shallow depth peak responses are considered, the maximum normalised $H$ and $M_{\text{LRP}}$ obtained from reinstallation near the idealised footprint cavity were 25% and 46% higher, respectively, than those obtained near the real footprint tests. The shallow depth peak values occurred at similar depths in both tests, $(H/As_n)_{\text{max}}$ occurred at $z/D=0.33$ and $(M/As_nD)_{\text{max}}$ occurred at $z/D=0.05$. However, the deep reinstallation response, that is, the reversed $H$ pushing the footing away from the footing centre and the moment bending the footing towards the footprint centre, were only applicable to the real footprint cases. The comparison reveals that soil heterogeneity has a greater effect on the deep reinstallation response than on the reinstallation response obtained at shallow depth.
Figure 8.17: Effect of Soil Heterogeneity on Jack-Up Reinstallation

Figure 8.18 compares the two rigid reinstallation tests conducted in two soil samples (sample BB1 and BB2). The trend of vertical, horizontal forces and moments exhibited similar patterns, with different peak values due to the variation in $s_u$ profile between the two soil samples. The vertical force of B-R-2 was observed to be lower than B-R-1 from $z/D=0.2$. This coincided with the depth of the soft layer observed from the $s_u$ profile of the soil sample BB2. Because the footing was penetrated in softer soil, the horizontal force and moment were lower.
8.4.5 Summary of Experimental Results

The maximum loadings and movements during shallow depth reinstallations are summarised in Table 8.6 and Table 8.7 respectively. The deep reinstallation maximum loadings and movements are summarised in Table 8.8 and Table 8.9. The sign convention and definition of terminologies are shown in Figure 1.8 and Figure 6.18. Detailed discussions and comparisons are presented in Section 8.4.6 to Section 8.4.8.
### Chapter 8: Physical Modelling of the Complete Jack-Up Reinstallation Problem

#### Table 8.6: Summary of Peak Responses from Beam Centrifuge Tests on Jack-up Reinstallations near Real Footprints (Shallow Depth Responses)

<table>
<thead>
<tr>
<th>Test Name</th>
<th>( H_{\text{max},s} ) (N)</th>
<th>( \left( \frac{H}{As_o} \right)_{\text{max},s} )</th>
<th>( (MLRP)_{\text{max},s} ) (Nm)</th>
<th>( \left( \frac{MLRP}{As_o D} \right)_{\text{max},s} ) (Nm)</th>
<th>( (MCP)_{\text{max},s} ) (Nm)</th>
<th>( \left( \frac{MCP}{As_o D} \right)_{\text{max},s} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-R-1</td>
<td>0.04</td>
<td>0.39</td>
<td>0.18</td>
<td>0.46</td>
<td>-0.93</td>
<td>-1.55</td>
</tr>
<tr>
<td>B-R-2</td>
<td>0.02</td>
<td>0.26</td>
<td>0.12</td>
<td>0.44</td>
<td>-0.35</td>
<td>-0.90</td>
</tr>
<tr>
<td>B-H-1</td>
<td>0.03</td>
<td>0.25</td>
<td>0.15</td>
<td>0.35</td>
<td>-0.73</td>
<td>-0.97</td>
</tr>
<tr>
<td>B-M-1</td>
<td>0.015</td>
<td>0.14</td>
<td>0.09</td>
<td>0.20</td>
<td>-0.31</td>
<td>-0.51</td>
</tr>
<tr>
<td>B-M-2</td>
<td>0.009</td>
<td>0.10</td>
<td>0.11</td>
<td>0.16</td>
<td>-0.19</td>
<td>-0.34</td>
</tr>
<tr>
<td>B-F-1</td>
<td>0.002</td>
<td>0.20</td>
<td>0.02</td>
<td>0.16</td>
<td>-0.07</td>
<td>-0.19</td>
</tr>
</tbody>
</table>

#### Table 8.7: Summary of Maximum Displacements from Beam Centrifuge Tests on Jack-up Reinstallations near Real Footprints (Shallow Depth Responses)

<table>
<thead>
<tr>
<th>Test Name</th>
<th>( h_{\text{max},s} ) (mm)</th>
<th>( \theta_{\text{max},s} ) (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-H-1</td>
<td>247.84</td>
<td>-</td>
</tr>
<tr>
<td>B-M-1</td>
<td>-</td>
<td>-0.18</td>
</tr>
<tr>
<td>B-M-2</td>
<td>-</td>
<td>-0.14</td>
</tr>
<tr>
<td>B-F-1</td>
<td>507.58</td>
<td>-0.79</td>
</tr>
</tbody>
</table>

#### Table 8.8: Summary of Peak Responses from Beam Centrifuge Tests on Jack-up Reinstallations near Real Footprints (Deep Reinstallation Response)

<table>
<thead>
<tr>
<th>Test Name</th>
<th>( H_{\text{max},d} ) (N)</th>
<th>( \left( \frac{H}{As_o} \right)_{\text{max},d} )</th>
<th>( (MLRP)_{\text{max},d} ) (Nm)</th>
<th>( \left( \frac{MLRP}{As_o D} \right)_{\text{max},d} ) (Nm)</th>
<th>( (MCP)_{\text{max},d} ) (Nm)</th>
<th>( \left( \frac{MCP}{As_o D} \right)_{\text{max},d} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-R-1</td>
<td>-0.09</td>
<td>-0.35</td>
<td>-0.69</td>
<td>-0.35</td>
<td>2.39</td>
<td>1.55</td>
</tr>
<tr>
<td>B-R-2</td>
<td>-0.07</td>
<td>-0.36</td>
<td>-0.55</td>
<td>-0.24</td>
<td>1.69</td>
<td>1.78</td>
</tr>
<tr>
<td>B-H-1</td>
<td>-0.06</td>
<td>-0.22</td>
<td>-0.66</td>
<td>-0.18</td>
<td>1.55</td>
<td>1.11</td>
</tr>
<tr>
<td>B-M-1</td>
<td>-0.06</td>
<td>-0.22</td>
<td>-0.41</td>
<td>-0.24</td>
<td>1.46</td>
<td>1.03</td>
</tr>
<tr>
<td>B-M-2</td>
<td>-0.05</td>
<td>-0.18</td>
<td>-0.48</td>
<td>-0.25</td>
<td>1.18</td>
<td>0.90</td>
</tr>
<tr>
<td>B-F-1</td>
<td>-0.04</td>
<td>-0.23</td>
<td>-0.29</td>
<td>-0.20</td>
<td>0.75</td>
<td>0.67</td>
</tr>
</tbody>
</table>

#### Table 8.9: Summary of Maximum Displacements from Beam Centrifuge Tests on Jack-up Reinstallation near Real Footprints (Deep Reinstallation Response)

<table>
<thead>
<tr>
<th>Test Name</th>
<th>( h_{\text{max},d} ) (mm)</th>
<th>( \theta_{\text{max},d} ) (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-H-1</td>
<td>20.97</td>
<td>-</td>
</tr>
<tr>
<td>B-M-1</td>
<td>-</td>
<td>0.54</td>
</tr>
<tr>
<td>B-M-2</td>
<td>-</td>
<td>1.25</td>
</tr>
<tr>
<td>B-F-1</td>
<td>251.34</td>
<td>0.21</td>
</tr>
</tbody>
</table>
8.4.6 Effect of Including Horizontal Flexibility

The response of a partially restrained jack-up leg during reinstallation near real footprint was investigated in phase one testing. Horizontal flexibility was prescribed in test B-H-1 simulating a jack-up leg connecting to the actuator with a horizontal spring. The results of B-H-1 were compared with rigid reinstallation test results B-R-1 which was conducted in the same soil sample. The comparisons are presented in Figure 8.19. The footing started to slide towards the footprint centre at touchdown level and the $h_{\text{max}}$ was around 250mm (h/D=0.04). With further penetration the footing moved backward and the final horizontal displacement was 21mm (h/D=0.0035) at z/D=1.5. The shallow depth $h_{\text{max}}$ was therefore more than ten times larger than the final horizontal displacement at deep reinstallation. The shallow depth reinstallation peak loadings, that is $H_{\text{max},s}$ and $(M_{LRP})_{\text{max},s}$ for B-H-1 were reduced by 25% and 15%, respectively, compared to the rigid reinstallation conditions. The peak loadings during deep reinstallation, $H_{\text{max},d}$ and $(M_{LRP})_{\text{max},d}$, were reduced by approximately 35% compared to the rigid reinstallation. The reduction ratio is more significant than the shallow depth reinstallation. However, the $H_{\text{max},d}$ and $(M_{LRP})_{\text{max},d}$ were still two times larger than the shallow depth values.
Figure 8.19: The Reinstallation Response of Jack-Up with Different Horizontal Flexibilities
The same three-stage responses could be observed in B-R-1 and B-H-1. As the footing came into contact with the founding soil, the soil reaction pushed the footing, and the footing moved rapidly in a horizontal direction towards the footprint centre. The $H$ continued to increase slowly compared to the rigid reinstallation case, reaching its peak value near the footprint toe ($z_F=0.33D$). This movement had a limited effect on the vertical force and the $M_{LRP}$. The vertical forces obtained for the two tests were similar. The $M_{LRP}$ increased from touchdown level and remained constant from $z=0.05D$ to $z=0.7D$. As penetration continued, the $H$ decreased, and the strong soil layer near $z_{FC}$ started to push the footing in the reverse direction. The footing then moved away from the footprint centre and almost returned to its original horizontal position. Near $z_{FC}$, the $M_{LRP}$ also acted in reverse direction, bending the footing towards the footprint centre.

For a jack-up leg partially restrained in the horizontal direction, when it was being reinstalled near real footprints, it slid towards the footprint centre at shallow depth and the jack-up leg moved back to almost its original horizontal position with further penetration. The $H_{\text{max,d}}$ and $M_{\text{max,d}}$ were 35% smaller than the case of rigid reinstallation.

### 8.4.7 Effect of Including Rotational Flexibility

The reinstallation response of jack-up leg partially restrained in angular direction was investigated. This simulated a hinge connection between the model jack-up leg and actuator and the results are shown in Figure 8.20. The jack-up leg initially rotated towards the footprint centre and the $\theta_{\text{max}}$ from B-M-1 and B-M-2 were 0.18° and 0.14°, respectively. With further penetration the jack-up leg rotated in reversed direction (away from footprint centre) and final angle of rotation from B-M-1 and B-M-2 were 0.54° and 1.25°, respectively. Although the shallow depth $\theta_{\text{max}}$ values from the two tests were similar, the final angle of rotation increased by more than two times when the rotational flexibility increased from 0.614°/MNm (B-M-1) to 1.35°/MNm (B-M-2). The change in rotation angle was directly proportional to the change in rotational flexibility. Figure 8.21 shows that the $\theta_{\text{max}}$ during deep reinstallation induced maximum equivalent horizontal displacement of 220mm ($h_T/D=0.04$) and 510mm ($h_T/D=0.08$) in the two tests (refer to Figure 7.16 for details of $h_T$).
Figure 8.20: The Reinstallation Response of Jack-Up with Different Rotational Flexibilities
The shallow depth peak loading values reduced with increasing rotational flexibility. The $(M_{CP})_{\text{max}}$ reduced by 12\% and 48\% in B-M-1 and B-M-2, respectively, compared to rigid reinstallation case (B-R-2). There were also 19\% and 25\% reductions in $H_{\text{max}}$ in the two tests, respectively, compared to B-R-2. The $(M_{LRP})_{\text{max}}$ in B-M-1 and B-M-2 were 13\% and 25\%, respectively, lower than B-R-2. The peak loadings during deep reinstallation were larger than the shallow depth reinstallation. The ratio of shallow depth to deep reinstallation $(M_{CP})_{\text{max}}$ was 0.19 and 0.14 in B-M-1 and B-M-2, respectively.

Allowing the jack-up leg to rotate did not change the responses trend, the same three-stage response was observed as for B-R-2. The jack-up leg started to rotate towards the footprint centre with a negative $M_{CP}$ at touchdown level. In the B-M-2 test, the footing rotated two ways near the touchdown level as the $M_{CP}$ fluctuated around zero with the load release. B-M-1 and B-M-2 reached maximum angles of rotation at $z/D=0.2$. As penetration continued below the footprint toe level, the highly varying soil strength near the footprint centre increased and pushed the footing in the reverse direction. The footing started to rotate away from the footprint centre and beyond its original position. The difference in the vertical responses of the three tests increased with increasing angular movement. The jack-up legs were installed into the soil at an angle with reduced project area, giving a lower vertical resistance.

In summary, for a jack-up leg partially restrained to rotation, during reinstallation it firstly rotated towards the footprint centre then rotated in the reverse direction with further penetration due to the influence of the strong soil layer near $z_{FC}$. Both the shallow depth and deep reinstallation peak loading values were lower than the rigid reinstallation scenario. However, the deep reinstallation peak loadings were larger than
the shallow depth values, and the footing rotated back and beyond its original position. The final $\theta_{\text{max}}$ values were greater than the shallow depth $\theta_{\text{max}}$ values. When the jack-up leg was allowed to slide or rotate during reinstallation near the real footprint, the trends of the shallow depth response were generally comparable to reinstallation near the idealised footprint cavity.

### 8.4.8 Effect of Including Cross-Coupled Flexibilities

In phase four testing, the structural properties of the jack-up unit was described by a full flexibility matrix (vertical-horizontal-rotational cross-coupled) at the CP and the model jack-up was reinstalled at 1.0D from a real footprint. Therefore, all three governing parameters: jack-up structural properties, footprint geometry and soil properties were investigated in test B-F-1. The results are presented in Figure 8.22.

The jack-up leg initially slid and rotated towards the footprint centre, the $h_{\text{max}}$ during shallow depth reinstallation was 508mm and the $\theta_{\text{max}}$ was 0.8°. With further penetration the jack-up leg then moved in reverse direction and the final horizontal distance (at $z/D=1.5$) was 251mm which was half of the movement at shallow depth. The final $\theta_{\text{max}}$ was 0.2° away from the footprint centre, giving a total horizontal displacement, $h_{\text{total}}$ of 158mm ($h_{\text{total}}/D=0.017$) at the footing level. The change of $h_{\text{total}}$ with penetration depth is shown in Figure 8.23. While the shallow depth peak loadings reduced to almost zero, the $H_{\text{max}}$ and $(M_{\text{CP}})_{\text{max}}$ were reduced by 43% and 16%, respectively, compared to the rigid reinstallation conditions.

The shallow depth $h_{\text{total}}$ was 0.14D ($h_{\text{total}}=832$mm), which was significantly larger than the allowable horizontal movement prescribed in SNAME (2002). The flexibility of the jack-up adopted in the test therefore considered the jack-up reinstallation response under extreme situation. Detailed developments of allowable movements are presented in Section 7.4.4.
Figure 8.22: The Reinstallation Response of Jack-Up with Cross-Coupled Horizontal and Rotational Flexibility
At touchdown level, the footing moved rapidly, sliding and rotating towards the footprint centre. The jack-up leg also rose up by 200mm. This negative vertical movement was induced from the relatively large vertical-moment coupling flexibility term ($C_{Mv}$) and the negative $M_{CP}$. As the footing rose up, the soil loading reduced rapidly and fluctuated around zero. As penetration continued, the loading at footing level gradually increased. The vertical movement resulting from the H and the $M_{CP}$ cancelled each other, and the jack-up leg penetrated the soil at an almost constant rate. The footing started to move in the reverse direction (sliding and rotating away from the footprint centre) for penetration below the footprint toe level. At $z/D=0.8$, the strong soil layer near $z_{FC}$ resulted in a relatively large negative H and positive $M_{LRP}$ and triggered movement in vertical direction. The jack-up leg dropped down by 300mm. As a result, the vertical, horizontal and moment loads increased rapidly. However, with further penetration, the horizontal force and moment gradually returned to their values before the reduction.

The coupling terms were all positive (different from flexibility matrix adopted in the scaled model test S-C-1, which had the coupling terms in opposite sign). With the opposite signed H and $M_{CP}$, the movements acted in opposite directions. Figure 8.24 shows the cancellation effect of the coupling terms. The decoupled displacement indicates that the H contributed 60% of the $h_{total}$.
By allowing the jack-up leg to slide, rotate and rise/penetrate, the shallow depth reinstallation peak response was eliminated as the horizontal force and moment decreased to almost zero. The footing initially slid and rotated towards the footprint centre. The stronger soil near \( z_{FC} \) helped to push the footing back to its original position. Although the loading still peaks near \( z_{FC} \), \( H_{max,d} \) and \( (M_{CP})_{max,d} \) were reduced by 43% and 16%, respectively, compared to the rigid reinstallation conditions.

In practice, if a jack-up could sustain the movement (and loading) during shallow depth reinstallation, with further penetration, the reverse loading at deep reinstallation could correct the movement back to almost its original position. Alternatively, a larger footing (or skirted footing) could be adopted to reduce the penetration depth requirement, this could avoid the large soil load near \( z_{FC} \).

### 8.5 CONCLUDING REMARKS

With the use of the real-time hybrid testing method, all of the three key parameters to the jack-up reinstallation problem, that is the effect of footprint geometry, soil heterogeneity and structural properties of jack-up unit were included, forming a complete model. A different three-stage response was observed during reinstallation near real footprints and the failure mechanism was highly influenced by the \( s_u \) profile near \( z_{FC} \). The key test results are summarised below:

- The shallow depth response was found to be similar to the tests involving the idealised footprint cavity indicating that the shallow depth response was controlled by the footprint geometry, rather than by the soil heterogeneity;
- The soil heterogeneity has a greater effect at deep reinstallation, the stronger soil layer near \( z_{FC} \) pushed the footing in the reverse direction, making the deep reinstallation loadings larger than in the shallow depth situation;
Chapter 8: Physical Modelling of the Complete Jack-Up Reinstallation Problem

- With the dual peak loading in reverse direction, the jack-up leg underwent two-ways movement when the jack-up leg was partially restrained;

- When the full jack-up model was considered, the jack-up slid and rotated towards the footprint centre at shallow depth. The shallow depth horizontal force and moment reduced to almost zero with movement towards the footprint centre. With further penetration the footing slid and rotated in reverse direction back to its original position;

- In practice, if the jack-up could sustain the movement (and loading) at shallow depth, the reverse loading at deep reinstallation could correct the movement nearly to its original position.
CHAPTER 9. INTERPRETATION OF THE EXPERIMENTAL RESULTS ON FLEXIBLE JACK-UP MODEL

9.1 INTRODUCTION

As demonstrated in Chapters 7 and 8, the reinstallation response changed with the structural properties of the jack-up. Movements of the model jack-up unit led to reductions of the horizontal force and moment. The purpose of this chapter is to assess the magnitude of the reduction in the peak loads and increase in movement with jack-up flexibility.

The comparison first considers the effect of flexibility in the one degree of freedom only (horizontal then rotational), before an appropriate method to combine multiple degrees of freedom into an overall jack-up flexibility is discussed and applied to normalise the different test results. Finally, the comparisons are extended to include the responses during deep reinstallation.

9.2 DEFINITION OF TERMINOLOGIES AND NORMALISATION OF PEAK RESPONSES

The structural properties of the jack-up were modelled by a 3x3 flexibility matrix at the common point, CP. The full 3x3 matrix was modified according to Section 7.2.2 and Section 8.2 for the four phases that were tested in a scale-model test at unit gravity and in a beam centrifuge.

To facilitate a comparison of the results, the reductions of maximum normalised loading (by percentage) were considered. The reductions in the maximum normalised loads were represented by the reduction ratio of the maximum normalised load from the reinstallation of the jack-up with finite flexibility to the maximum normalised load from the rigid reinstallation test (model jack-up leg fully fixed to the actuator). This concept is expressed in the following equation:

\[
\text{Reduction} = \left( \frac{\text{Maximum Normalised Load (Rigid)} - \text{Maximum Normalised Load (Flexible)}}{\text{Maximum Normalised Load (Rigid)}} \right) \times 100 \quad (9.1)
\]

Except in Section 9.5.2, the maximum loads discussed in this chapter refer to the shallow depth peak response. The maximum normalised horizontal force \((H/s_{uA})_{\text{max}}\) and maximum normalised moment \((M/s_{uA}D)_{\text{max}}\) refer to the loading at the load reference point (LRP). For the case of rigid reinstallation, the reductions in \((H/s_{uA})_{\text{max}}\)
and \((M/s_uAD)_{\text{max}}\) are equal to zero. Larger reductions in \((H/s_uA)_{\text{max}}\) and \((M/s_uAD)_{\text{max}}\)
correspond to larger differences between the reinstallation response from the jack-up
with finite flexibilities and the rigid reinstallation.

The trends in the reductions in \((H/s_uA)_{\text{max}}\) and \((M/s_uAD)_{\text{max}}\) with respect to the
flexibility (at prototype scale) were established to quantify the effect of the structural
properties.

It is often in the industry’s interest to assess the possible jack-up movements during
reinstallation. Therefore, the change in the maximum movements with flexibilities was
also studied. The maximum movements were studied in terms of the normalised
maximum horizontal displacement at the LRP \((h_{\text{max}}/D)\) and the rotation at the CP \(\theta_{\text{max}}\)
in degree).

### 9.3 HORIZONTAL FLEXIBILITY

When finite horizontal flexibility \((C_{Hh})\) was specified at the CP of the jack-up, the
model jack-up leg could only slide horizontally, which represented a horizontal spring
connection between the model jack-up leg and the actuator. The effect of horizontal
flexibility was quantified by comparing the reductions in the maximum normalised
loads \((H/s_uA)_{\text{max}}\) and \((M/s_uAD)_{\text{max}}\). The shallow depth peak responses during
reinstallation near idealised footprint cavities (in scaled model test) and near real
footprints (in beam centrifuge) are presented together in Figure 9.1.

The result of 0% is of course the rigid installation and the dashed line a possible trend,
though more experimental data is required to validate it. A general trend of non-linear
reduction in peak response with increasing horizontal flexibility is observed. The peak
response decreased significantly with the inclusion of relatively low horizontal
flexibility. However, the reduction decayed, and there was no further reduction in the
peak response with a further increase in horizontal flexibility. For the range of jack-up
flexibility considered in this study, the maximum reductions in \((H/s_uA)_{\text{max}}\) and
\((M/s_uAD)_{\text{max}}\) were approximately 45% and 30%, respectively. Although the soil load
was released as the footing slid towards the footprint centre, the vertical soil load also
became more eccentric and induced a moment in the footing. Therefore, allowing the
jack-up unit to slide caused a larger reduction in the peak horizontal force than in the
moment.

Although the reductions in the peak response were highly non-linear, the maximum
horizontal displacement in the jack-up leg \((h_{\text{max}})\) increased linearly with the horizontal
flexibility. Because the largest \(h_{\text{max}}\) considered in this study \((h_{\text{max}}/D=0.155)\) already
exceeds the allowable limit in the field \((h/D=0.078,\) refer to Section 7.4.4 for detail), the
linear relation established here is sufficiently applicable to a wide range of load-
displacement responses.
Chapter 9: Interpretation of the Experimental Results on Flexible Jack-up Model

9.4 ROTATIONAL FLEXIBILITY

Only finite rotational flexibility ($C_{Mo}$) was prescribed at the common point, CP, to model a hinge connection between the model jack-up leg and the actuator. The effect of rotational flexibility on the responses during shallow depth reinstallation is shown in Figure 9.2.
Similar to the case of horizontal flexibility, (H/s_uA)_{max} and (M/s_uAD)_{max} reduced non-linearly with increasing rotational flexibility. The maximum reduction in (H/s_uA)_{max} was approximately 55%. However, the reduction in (M/s_uAD)_{max} was relatively scattered, and the reduction was more significant in the case of reinstallation near real footprints than near idealised footprint cavities. The angular rotation also increased linearly with the rotational flexibility.
During reinstallation near idealised footprint cavities, the moment was reduced by the angular movement. However, during reinstallation near real footprints, the footings were rotated into the remoulded soil around the real footprint; the lower soil strength induced a further reduction in the moment. Therefore, including the rotational flexibility in the modelling of jack-up reinstallation response has a greater effect on reinstallation near real footprints (subjected to the combined footprint geometric effect and soil heterogeneity) than on reinstallation near idealised footprint cavities (only the footprint geometric effect is included). In both cases of reinstallation near idealised footprint cavities and real footprints, the peak responses became insensitive to a further increase in rotational flexibility when the rotational flexibility was relatively large.

### 9.5 COMBINED JACK-UP FLEXIBILITY

The effect of the jack-up structural properties on the reinstallation response was further assessed by comparing the experimental results from the jack-up model with different types of flexibilities. To facilitate the comparison, the 3x3 flexibility matrix was converted into a single value, which is defined as the equivalent combined flexibility, $C_{\text{combined}}$. $C_{\text{combined}}$ is defined as:

$$
C_{\text{combined}} = \sqrt{C_{\text{hlh}}^2 + (C_{\text{Mh}}D)^2 + 2C_{\text{hlh}}C_{\text{Mh}}D + (C_{\text{H0L}})^2 + (C_{\text{Mh}}DL)^2 + 2C_{\text{H0L}}C_{\text{Mh}}DL} \quad (9.2)
$$

The mathematical development of $C_{\text{combined}}$ is presented in Figure 9.3. Based on the sign convention described in Figure 1.5, positive horizontal force and positive moment would result in movement in opposite direction. Therefore, the moment was multiplied by a negative sign so that the direction of movement from the horizontal force component and from the moment component could be consistent.
Chapter 9: Interpretation of the Experimental Results on Flexible Jack-up Model

Figure 9.3: Derivation of Equivalent Combined Flexibility

Original 3x3 Flexibility Matrix:

\[
\begin{bmatrix}
C_{lh} & 0 & C_{Mb}\ H \\
C_{hv} & 0 & C_{Mv} \\
C_{1h} & 0 & C_{Mb} \ M
\end{bmatrix} = \begin{bmatrix}
h \\
0 \\
0 \\
\end{bmatrix}, \text{ units of matrix:} \begin{bmatrix}
\text{mm} / \text{N} & 0 & \text{mm} / \text{Nm} \\
\text{mm} / \text{N} & 0 & \text{mm} / \text{Nm} \\
\text{rad} / \text{N} & 0 & \text{rad} / \text{Nm} \end{bmatrix} \begin{bmatrix} N \\ 0 \\ Nm \end{bmatrix} = \begin{bmatrix} mm \\ 0 \\ mm \end{bmatrix}
\]

Convert the Matrix

Convert the matrix so that
1) Convert the matrix so that all the terms are with the same unit
2) Convert the matrix \( h \) and \( qL \) being positive for movement towards the footprint centre

\[
\begin{bmatrix}
C_{lh} & 0 & C_{Mb} \ D \\
C_{hv} & 0 & C_{Mv} \ D \\
C_{1h} \ L & 0 & C_{Mb} DL \end{bmatrix} = \begin{bmatrix}
h \\
0 \\
- \frac{M}{D} \end{bmatrix}, \text{ units of matrix:} \begin{bmatrix}
\text{mm} / \text{N} & 0 & \text{mm} / \text{N} \\
\text{mm} / \text{N} & 0 & \text{mm} / \text{N} \\
\text{mm} / \text{N} & 0 & \text{mm} \end{bmatrix} \begin{bmatrix} N \\ 0 \\ Nm \end{bmatrix} = \begin{bmatrix} mm \\ 0 \\ mm \end{bmatrix}
\]

Expand the Matrix:

Expand the matrix and consider the horizontal and rotational load-displacement relationships

\[
h = C_{1h} \ H + \left( - \frac{M}{D} \ C_{Mb} \ D \right)
\]

\[
0L = C_{1h} \ LH + \left( - \frac{M}{D} \ C_{Mb} DL \right)
\]

Combine Horizontal and Rotational Movement:

Combine the effect of the various flexibility terms by considering the resultant movements:

\[
h_1 = \sqrt{h^2 + (0L)^2} = \left( C_{1h} H \right)^2 + \left( - \frac{M}{D} \ C_{Mb} D \right)^2 + 2 \left( C_{1h} \ C_{Mb} D H \right) \left( - \frac{M}{D} \right) + 2 \left( C_{1h} \ C_{Mb} \ D L \right) \left( - \frac{M}{D} \right)^2
\]

\[
(0L)^2 = \left( C_{1h} LH \right)^2 + \left( - \frac{M}{D} \ C_{Mb} DL \right)^2 + 2 \left( C_{1h} \ C_{Mb} DL H \right) \left( - \frac{M}{D} \right) + 2 \left( C_{1h} \ C_{Mb} \ DL^2 \right) \left( - \frac{M}{D} \right)^2
\]

\[
h_2 = \sqrt{\left( C_{1h} H \right)^2 + \left( - \frac{M}{D} \ C_{Mb} D \right)^2 + 2 \left( C_{1h} \ C_{Mb} D H \right) \left( - \frac{M}{D} \right) + \left( C_{1h} LH \right)^2 + \left( - \frac{M}{D} \ C_{Mb} \ DL \right)^2 + 2 \left( C_{1h} \ C_{Mb} \ DL H \right) \left( - \frac{M}{D} \right) + 2 \left( C_{1h} \ C_{Mb} \ DL^2 \right) \left( - \frac{M}{D} \right)^2}
\]

Consider a Unit Load:

Simplify the equation by considering a unit horizontal and moment load

\[
h_1 = \sqrt{\left( C_{1h} H \right)^2 + \left( - \frac{M}{D} \ C_{Mb} D \right)^2 + 2 \left( C_{1h} \ C_{Mb} D H \right) \left( - \frac{M}{D} \right) + \left( C_{1h} LH \right)^2 + \left( - \frac{M}{D} \ C_{Mb} \ DL \right)^2 + 2 \left( C_{1h} \ C_{Mb} \ DL H \right) \left( - \frac{M}{D} \right) + 2 \left( C_{1h} \ C_{Mb} \ DL^2 \right) \left( - \frac{M}{D} \right)^2}
\]

\[
h_1 \propto \sqrt{\left( C_{1h} H \right)^2 + \left( C_{Mb} D \right)^2 + 2 \left( C_{1h} \ C_{Mb} D H \right) \left( C_{1h} \ L \right) + \left( C_{Mb} \ DL \right)^2 + 2 \left( C_{1h} \ C_{Mb} \ DL H \right) \left( C_{1h} \ L \right) + 2 \left( C_{1h} \ C_{Mb} \ DL^2 \right) \left( C_{1h} \ L \right)^2}
\]

Define \( C_{\text{combined}} \):

\[
C_{\text{combined}} = \sqrt{\left( C_{1h} H \right)^2 + \left( C_{Mb} D \right)^2 + 2 \left( C_{1h} \ C_{Mb} D H \right) \left( C_{1h} \ L \right) + \left( C_{Mb} \ DL \right)^2 + 2 \left( C_{1h} \ C_{Mb} \ DL H \right) \left( C_{1h} \ L \right) + 2 \left( C_{1h} \ C_{Mb} \ DL^2 \right) \left( C_{1h} \ L \right)^2}
\]
Table 9.1 and Table 9.2 show the $C_{combined}$ (in prototype scale) and the type of flexibility matrices adopted in 1g-scaled model tests and in beam centrifuge tests, respectively.

### Table 9.1: $C_{combined}$ of the Flexibility Matrices Adopted in 1g-Scaled Model Tests.

<table>
<thead>
<tr>
<th>1g-Scaled Test</th>
<th>Horizontal Flexibility</th>
<th>Rotational Flexibility</th>
<th>Cross-Coupled Flexibility</th>
<th>$C_{combined}$ (m/kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-H-1</td>
<td>✓</td>
<td>✓</td>
<td>-</td>
<td>0.015</td>
</tr>
<tr>
<td>S-H-2</td>
<td>✓</td>
<td>-</td>
<td>-</td>
<td>0.055</td>
</tr>
<tr>
<td>S-M-1</td>
<td>-</td>
<td>✓</td>
<td>-</td>
<td>0.013</td>
</tr>
<tr>
<td>S-M-2</td>
<td>-</td>
<td>✓</td>
<td>-</td>
<td>0.114</td>
</tr>
<tr>
<td>S-L-1</td>
<td>✓</td>
<td>✓</td>
<td>-</td>
<td>0.020</td>
</tr>
<tr>
<td>S-C-1</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>0.033</td>
</tr>
</tbody>
</table>

### Table 9.2: $C_{combined}$ of the Flexibility Matrices Adopted in Beam Centrifuge Tests.

<table>
<thead>
<tr>
<th>Beam-Centrifuge Test</th>
<th>Horizontal Flexibility</th>
<th>Rotational Flexibility</th>
<th>Cross-Coupled Flexibility</th>
<th>$C_{combined}$ (m/kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-H-1</td>
<td>✓</td>
<td>-</td>
<td>-</td>
<td>0.008</td>
</tr>
<tr>
<td>B-M-1</td>
<td>-</td>
<td>✓</td>
<td>-</td>
<td>0.015</td>
</tr>
<tr>
<td>B-M-2</td>
<td>-</td>
<td>✓</td>
<td>-</td>
<td>0.033</td>
</tr>
<tr>
<td>B-F-1</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>0.041</td>
</tr>
</tbody>
</table>

The horizontal displacement and rotation of the jack-up unit were also combined by determining $h_{total}$ at the footing load reference level (LRP). The concept of $h_{total}$ is presented in Figure 7.16 and the detailed equation is shown below:

$$h_{total} = \max\left(\sqrt{L \tan(\theta)} + h\right)$$  \hspace{1cm} (9.3)

#### 9.5.1 Effect of Combined Flexibilities on Peak Response

Figure 9.4 shows the reinstallation response of the jack-up unit with different types of flexibilities. $(H/s_uA)_{max}$ and $(M/s_uAD)_{max}$ decreased non-linearly with an increase in $C_{combined}$. The non-linear relationship observed here confirmed that the peak responses did not decrease infinitely to 100% reduction with increases in the jack-up flexibility.

Two trend lines were established to represent the upper and lower trends between the reductions in $(H/s_uA)_{max}$ and $(M/s_uAD)_{max}$ with $C_{combined}$. Whereas the upper trend line fits the results from the reinstallation near an idealised footprint cavity, the lower trend line was established for the reinstallation near a real footprint.
As discussed in Section 9.4, the reduction in the peak moment responses during reinstallment near an idealised footprint cavity was smaller than near a real footprint due to the combined effect of footprint geometry and lower strength of the soil around the footprint. The effect of soil heterogeneity is difficult to assess as it is related to the strength of the undisturbed soil, the soil stress history, the depth of penetration, the
preloading vertical load level, the offset distance, the operation time (OT) and the elapsed time (ET) between jack-up visits (see for instance Gan, 2010).

The upper trend line is therefore recommended for a conservative estimate of the reductions in \((H/s_uA)_{\text{max}}\) and \((M/s_uAD)_{\text{max}}\). If the effect of soil heterogeneity could be assessed by confirming the reduction in the strength of the soil around the footprint by means of laboratory testing or in-situ testing, for example, then the lower trendline could be adopted.

Figure 9.4 also shows the normalised maximum horizontal displacement and maximum angular movement. For the case of horizontal displacement, a single linear trend line fits both the results from reinstallation near idealised footprint cavities and real footprints. However, the results on maximum angular movements were relatively scattered. To better estimate the jack-up movement during jack-up reinstallation, a further assessment was conducted on the change in the total horizontal displacement with jack-up flexibility. Figure 9.5 shows that a linear upper and lower trend lines could be established between \(h_{\text{total}}\) and \(C_{\text{combined}}\). The lower trend line is only applicable for the movement of the model jack-up associated with a hinge connection to the actuator. The upper trend line fits the responses of the jack-up with a different type of flexibility and is therefore recommended for a conservative prediction of jack-up movement.

![Figure 9.5: The Effect of Jack-Up Structural Flexibility on the Total Horizontal Displacement](image)
9.5.2 Effect of Combined Flexibility on Deep Reinstallation Response

All of the previous comparisons considered the shallow depth reinstallation response from 1g scale-model tests and beam centrifuge tests. The effect of the jack-up structural stiffness on the deep reinstallation response was investigated by examining the beam centrifuge test results in detail here. Figure 9.6 and Figure 9.7 compare the reductions in $(H/s_uA)_{max}$ and $(M/s_uAD)_{max}$ during shallow and deep jack-up reinstallations. A non-linear relationship was found between the reduction in the peak responses and $C_{combined}$. However, the rates of reduction of $(H/s_uA)_{max}$ and $(M/s_uAD)_{max}$ during the deep reinstallation were less than during the shallow depth reinstallation (refer to Section 8.4.2 for detail). Therefore, another trend line was established for the prediction of deep reinstallation response. Similarly, two trend lines were established to fit the relatively scattered data on jack-up movement from shallow depth reinstallation and deep reinstallation. Both the maximum horizontal displacement and angular movement increased linearly with $C_{combined}$. 
Figure 9.6: The Effect of Jack-Up Structural Flexibility on the Peak Responses during Shallow and Deep Reinstallation

Figure 9.7 shows that $h_{\text{total}}$ during deep reinstallation was less than the shallow depth movement because the footing experienced movement in two directions (refer to Section 8.4 for details). In theory, if the jack-up could sustain the movement and loading that occurred during shallow depth reinstallation, the reversed movement during deep reinstallation would help to restore the jack-up unit back to its original position. However, the magnitude of the reversed movement during deep reinstallation was not
only related to the shallow depth movement but also to the jack-up structural stiffness. For the two tests that used a jack-up model with a hinge connection, the reversed movement was so large that \( h_{\text{total}} \) during deep reinstallation reduced into the negative region. This indicates that the footing moved away from the footprint centre. In the majority of cases, movement towards the footprint centre is the critical movement direction because, as the jack-up moves closer to the footprint centre, it could collide with the platform located near the footprint. Therefore, because of the complexity of the deep reinstallation response, including the possible advantages from the reversed movement in the prediction is not recommended. The maximum jack-up movement should be conservatively evaluated from the shallow depth reinstallation response presented in Figure 9.5.

![Figure 9.7: The Effect of Jack-Up Structural Flexibility on the Peak Total Horizontal Displacement during Shallow and Deep Reinstallation](image)

The difference between the shallow and deep reinstallation responses could be related to the difference in the embedment depth. This concept is illustrated in Figure 9.8. During shallow depth reinstallation, the footing was only partially buried within the soil. With a relatively small movement of the footing, the soil load around the footing was released, and the peak responses reduced significantly. However, during deep reinstallation, the footing was completely buried within the soil. Instead of a point load acting at the free end of the model jack-up leg, the relatively large soil load restrained footing movement. This reduced the actual flexibility of the reinstalled jack-up leg.
Therefore, smaller reductions in the peak responses and in the movement of the footing were observed.

The difference in the governing parameters of the shallow and deep reinstallations could also contribute to the difference in the responses. The shallow depth reinstallation response was controlled by the footprint geometry. Footing movement towards the footprint centre reduced the offset distance, which effectively reduced the peak reinstallation load. Therefore, the shallow depth reinstallation response was relatively sensitive to changes in the jack-up structural properties. However, during deep reinstallation, the peak responses were controlled by the soil heterogeneity. Although the footing moved away from the footprint centre, the footing was still surrounded by highly remoulded soil that imposed a relatively large horizontal force and moment on the footing. Therefore, the structural properties of the jack-up unit have a greater effect on the shallow depth reinstallation response than on the deep reinstallation response.

9.6 CONCLUDING REMARKS

In this study, reinstallation test results for a jack-up with different flexibilities were compared to improve knowledge of the effect of jack-up structural properties on the peak reinstallation response.
The shallow depth peak responses reduced non-linearly with increasing horizontal or rotational flexibility. However, in the relatively high flexibility range, the decrease in the peak loading decayed and became insensitive to any further increases in the jack-up flexibility. The 3x3 flexibility matrices were also normalised into a single flexibility representing an equivalent combined flexibility of the jack-up. This allowed comparison between the reinstallation responses of the jack-up with different types of flexibility. Further comparison was made between the shallow depth and deep reinstallation responses. The shallow depth reinstallation response was more sensitive than the deep reinstallation response to changes in the equivalent combined flexibility.

An upper trend line for the reduction in the maximum normalised horizontal force and moment were established for an initial estimate of the effect of the jack-up structural properties on the shallow depth reinstallation response. The reinstallation load can firstly be estimated using the simple prediction method presented in Chapter 5, the peak loading can then be adjusted for the jack-up flexibility according to the trend lines established in this chapter. In addition, the trend of the change in the maximum movement associated with a change in the equivalent combined flexibility was established. This allowed an initial estimate to be made of the direction and magnitude of jack-up movement during reinstallation. This would be particularly useful for the case of jack-up workover, where the jack-up is located in close proximity to the existing platform.

The purpose of this study was to improve the understanding on the role of jack-up structural properties and allow for an initial estimation of the peak reinstallation loads. Therefore, only two jack-up units and two different clay soils were studied in the reinstallation tests. Further experimental investigation is required to confirm the validity of the relationships for the prediction of the reinstallation response of a wider range of jack-up units in more diversified soil types.
CHAPTER 10. CONCLUSIONS

10.1 INTRODUCTION

This study investigated the response of a jack-up unit installed nearby existing footprints. The response is complex and the investigation was therefore conducted by considering the three governing parameters identified (footprint geometry, soil heterogeneity and jack-up structural properties) first in isolation and then in combination. The major achievements of this study are: 1) the identification the role of footprint geometry and the critical geometric parameters in the reinstallation response; 2) the development of a simple method for the prediction of vertical, horizontal and moment loads for shallow depth reinstallation up to two times of the footprint toe level; 3) the development of a real-time hybrid testing method for the testing of a full jack-up unit, 4) the identification of reinstallation response of a jack-up with different structural properties (flexibility).

These findings have been achieved through comprehensive experimental investigations on the laboratory floor and in a drum and beam centrifuge. The key findings are summarised in this chapter, together with a discussion on the limitations. Finally an opinion on future research opportunities is presented.

10.2 MAIN FINDINGS

10.2.1 Effect of Footprint Geometry to Jack-Up Reinstallation Response

In order to investigate the isolated effect of footprint geometry, idealised footprint cavities in conical shape were considered in the reinstallation tests. A V-shape blade was used to remove the soil and this left a seabed depression without remoulding the soil. Three different sizes of idealised footprints with footprint depth ranged from 0.167D to 0.67D were considered and the geometries covered the upper and lower bound of the real footprint profiles previously studied in a geotechnical centrifuge. A total of 16 reinstallation tests near an idealised footprint cavity were conducted in the UWA drum centrifuge. The experimental results were consistent. While the peak moment occurred near the touchdown level, the peak horizontal force occurred near the footprint toe level. The trend of response and the peak loadings from reinstallations near idealised footprint cavities were comparable with previous studies considering real footprints. This indicated that the footprint geometry governed the response during shallow depth reinstallation. The reinstallation response was also found to be sensitive to the offset distance, footprint size and diameter ratio.
Chapter 10: Conclusions

The soil failure mechanisms governing the change in reinstallation response with footprint geometry was also investigated by half-footing reinstallation tests and PIV techniques. During reinstallation, the soil failed in a skewed Hill-type mechanism or a slope failure mechanism. The footing was therefore subjected to eccentric vertical load and this resulted in moment. The eccentricity was largest near the touchdown level and gradually reduced with penetration depth and became almost zero near footprint toe level. The failed soil block moving horizontally, together with unbalanced lateral earth pressure on the two sides of footing, imposed horizontal load to the footing.

While the moment during the reinstallation of skirted footing was comparable to the flat-base footing, the horizontal force of skirted footing was three times larger than flat-base footing. The larger footing thickness induced a larger difference in lateral earth pressure on the two side of the skirted footing, therefore, giving a larger horizontal force. The benefit of the use of skirted footing is therefore inconclusive and should be further assessed by considering the loading and movement together.

The reinstallation response near an idealised footprint cavity was similar to reinstallation near a real footprint with relatively long elapsed time. This is because the remoulded soil around the footprint regained its strength with the relatively long elapsed time and therefore the reinstallation was dominated by the geometric effect rather than the soil heterogeneity effect. These experimental results provided an improved understanding on the isolated geometric effect of footprint and provided a basis for the development of prediction method for shallow depth reinstallation response.

10.2.2 Prediction Method for Shallow Depth Reinstallation Response

In this study, a simple method predicting the reinstallation response at different offset distance from different footprint size was developed. It is a significant improvement over the current industry guidelines SNAME (2002) which only provide suggestion on the minimum offset distance. The simple method is capable in predicting vertical, horizontal and moment load during jack-up reinstallation from touchdown level to two times footprint toe level. There are only two key input parameters to the method: the strength of undisturbed soil and the geometry of footprint. The method was calibrated using the results from footing reinstallation test near idealised footprint cavities.

It was proposed to predict the vertical force by modifying the bearing capacity equations from Hossain (2008) with a reduction factor (RF) to incorporate the geometric effect of the footprint. The RF at touchdown level was determined from three-dimensional finite element analysis. A design chart for RF was presented for footing reinstallation near a footprint with different factors of safety against slope failure. The moment was predicted from the maximum eccentricity, \( (e/D)_{\text{max}} \) measured from the reinstallation tests. The \( (e/D)_{\text{max}} \) generally reduced with offset distance but was insensitive to change in footprint size. Similarly, the horizontal force was predicted...
from the measured inclination angle. A simplified relationship between the inclination angle and footprint size was presented. The prediction method was validated by comparing with two other experimental investigations on footing reinstallation near a real footprint. The method generally over-estimated the horizontal force and moment but the trends of predicted change of loads with depth were in excellent agreement with test results.

The prediction method was developed for a fully-fixed footing (no sliding or rotation) and the predicted loading would therefore be conservative. In view of the complexity and often lack of detailed information on the reinstallation problem, the over-estimation provides the necessary safety margin to the prediction method.

The proposed prediction method provides a complete reinstallation profile for shallow depth jack-up reinstallation response. For reinstallation of jack-up with a relatively short elapsed time or for penetration near \( z_{FC} \), the responses are also influenced by the effect of soil heterogeneity. In such cases, it is recommended that the response should be investigated by a site specific physical and/or numerical modelling of the problem.

### 10.2.3 Real-Time Hybrid Testing for Modelling of Jack-Up Reinstallation

Although known to be a governing parameter, the effect of structural properties of a full jack-up unit to the reinstallation responses had yet to be investigated. It was because of the limitation of testing equipment that the jack-up unit was assumed to be fully rigid at the leg-hull connection level, ignoring the finite structural flexibility of the jack-up unit. In this study, the real-time hybrid testing method pioneered in structural dynamic testing was extended for modelling a full jack-up unit. This study therefore provided the first study investigating the reinstallation response of a full jack-up unit.

The reinstalling jack-up leg, which is the critical element, was modelled physically while the rest of the jack-up unit (the hull and the two pre-embedded jack-up legs) was modelled numerically. The physical model and the numerical model were connected at a common point (which was located on the reinstalling jack-up leg). A real-time control algorithm was constructed so that the physical model interacted with the numerical model in real-time, simulating the response of a full jack-up model.

A new vertical-horizontal-rotational actuator (VHM actuator) was developed to slide or rotate the physical jack-up model into or away from the footprint during reinstallation. This is the first such VHM actuator developed for the UWA centrifuge, and is only one of a handful worldwide (Dean et al. 1997, Punrattansin et al. 2003)

Within the hybrid testing, the numerical model was simplified into a 3x3 flexibility matrix at the common point. Jack-up units with different flexibility could therefore be modelled easily by updating the flexibility matrix. The following assumptions were made to reduce the complexity of the problem.
Chapter 10: Conclusions

- The vertical load-displacement cross-coupling terms $C_{Vh}$, $C_{Vv}$ and $C_{V\theta}$ were eliminated and the installation of the jack-up leg was modelled by the application of a constant vertical displacement step $\Delta v_{\text{step}}$;

- The current real-time hybrid testing method considered constant jack-up flexibility throughout reinstallation. The non-linear jack-up load-displacement response due to the P-$\Delta$ were therefore ignored;

- In theory, as the physical model leg penetrates into the soil, the length of the jack-up leg in the numerical model must increase by the same length. This change in jack-up leg length (therefore jack-up flexibility) was not considered in the numerical model.

These assumptions were necessary for the development of a relatively simple numerical model. However, the test results still provided valuable information to the effect of jack-up structural properties to the reinstallation response. The numerical model could be modified to update jack-up flexibility in real-time in a future study.

The use of hybrid testing is uncommon within geotechnics, with only the studies of soil liquefaction and dynamically loaded piles found in the literatures. This thesis provides detailed steps on how to apply this powerful and novel modelling technique in a wider range of geotechnical problem in the future.

10.2.4 Response of a Full Jack-Up during Reinstallation

Jack-up units with finite flexibility were modelled using the real-time hybrid testing method. The shallow depth reinstallation response was investigated by reinstalling a scaled jack-up model near an idealised footprint cavity under unit gravity. The model jack-ups were reinstalled at the most critical offset distance ($\beta=1.0D$) and for the most realistic TB footprint geometry.

The tests were conducted in three phases with load-displacement relationship in increasing complexity:

Phase 1: Single axis linear flexibility to model a horizontal spring and a hinge connection separately;

Phase 2: Linearly combined horizontal and rotational flexibilities to model an idealised jack-up with combined horizontal spring and hinge connection between the model jack-up leg and actuator;

Phase 3: Cross-coupled horizontal and rotational flexibilities to model the actual structural stiffness of a full jack-up unit.

The results from phase 1 tests shown that the peak loadings generally reduce with increasing jack-up flexibility and the jack-up slid or rotated towards the footprint centre.
There were significant differences between the responses of the jack-up with linearly combined horizontal and rotational flexibility matrix (idealised) and the full flexibility matrix with cross-coupled horizontal and rotational flexibilities (actual jack-up stiffness). When linearly combined horizontal and rotational flexibilities were adopted, the footing slid toward and rotated away from the footprint centre at the same time without significant change to the peak loadings (in terms of $H_{\text{max}}$ and $(M_{\text{LRP}})_{\text{max}}$). A full jack-up unit was considered with the implementation of cross-coupled horizontal and rotational flexibilities. The model jack-up leg slid and rotated towards the footprint centre at the same time. $H_{\text{max}}$ and $(M_{\text{LRP}})_{\text{max}}$ were reduced by 65% and 30%, respectively, compared to the rigid reinstallation test. Therefore, it is important to consider the full flexibility matrices with cross-coupled horizontal and rotational flexibility for realistic assessment of jack-up reinstallation response.

The jack-up reinstallation response near real footprints was studied by real-time hybrid testing in the beam centrifuge. All three governing parameters: footprint geometry, soil heterogeneity and jack-up structural properties were considered in this complete model. The reinstallation response was highly influenced by the $s_u$ profile near the maximum penetration depth of the footprint creation ($z_{\text{FC}}$). The model jack-up slid and rotated towards the footprint centre at shallow depth, with $h_{\text{max}}/D=0.085$ and $\theta_{\text{max}}=0.8^\circ$ and the horizontal force and moment reducing to almost zero. With further penetration, a stronger layer of soil near $z_{\text{FC}}$ pushed the footing in the opposite direction, giving a relatively large load in the reverse direction. Indeed the footing almost moved back to its original position during the deep reinstallation.

The effect of jack-up structural properties to the reinstallation response was further quantified by comparing the reduction in maximum normalised load with equivalent combined flexibility. The 3x3 matrix describing the flexibility of a jack-up unit was converted into a single parameter of equivalent combined flexibility. The maximum normalised load reduced non-linearly with increasing equivalent combined flexibility and trend lines were established for the prediction of reduction in peak horizontal force and moment. Eventually the reinstallation response became insensitive to further increase in jack-up flexibility and reduction limits were observed. The suggested upper limit to the reduction in normalised horizontal force and moment were 95% and 65% respectively. A linear relationship was found between the total horizontal displacement at the footing and equivalent combined stiffness. The effect of jack-up structural properties was more significant in the shallow depth than in deep reinstallation near $z_{\text{FC}}$. Both the reduction in peak loading and the footing movement were less sensitive to the change in equivalent combined flexibility during deep reinstallation. Therefore, separate lower trend lines were established for the prediction of deep reinstallation response.

Based on the test results, design charts were also developed to correct the peak loadings predicted from the simple prediction method for flexible jack-up unit with different equivalent combined flexibility.


10.3 **FURTHER RESEARCH**

10.3.1 **Refinement of the Simple Prediction Method**

The accuracy of the simple prediction method could be enhanced by refining the design charts for the key factors including \((c/D)_{\text{max}}\), \(\alpha_{\text{max}}\), \(a_{\text{M}}\) and \(a_{\text{H}}\) with more experimental data. Additional reinstallation tests in the centrifuge could consider a wider range of offset distance and footprint sizes and this would improve the accuracy of the prediction method. Furthermore, the prediction method was developed based on a flat-base footing on one type of soil. Additional investigation considering different footing geometry (e.g. conical base) and clay soil with different strengths is important for the verification of the simple prediction method. Calibration with in-situ data would also be of interest.

10.3.2 **Deep Reinstallation Response**

A more detailed experimental investigation is recommended with focus on the effect of the strong soil layer near \(z_{\text{FC}}\) on the reinstallation response. With the additional data, the effect of soil heterogeneity could be incorporated into the prediction method suggested in this study by adding another reduction factor \(RF_s\) to Equation (5.2). The profile of change in \(RF_s\) with depth should vary with \(z_{\text{FC}}\). \(RF_s\) could change from positive to negative depending on the stress history of undisturbed soil and the operation procedures of the previous jack-up unit.

10.3.3 **Further Investigation on the Effect of Structural Properties**

Most of previous experimental studies considered jack-up leg with zero flexibility at the leg-actuator connection (fully-fixed connection). This study is the first to consider finite jack-up flexibility in the reinstallation response. However, only two jack-up units and configurations were considered. Investigation of more jack-up unit dimensions and configurations will confirm the trend in the change of peak responses and movement with equivalent combined flexibility. The effect of structural flexibility could be investigated in details by considering the isolated effect of change in jack-up leg stiffness and foundation stiffness. Through this the critical structural properties to the reinstallation response could be identified. Furthermore, other footprint geometries, such as other footprint type TA and TC and reinstallation at offset distance other than 1.0D could also be considered. This would improve the understanding of the effect of footprint geometry to a jack-up with finite flexibility.
10.3.4 Development of Large Deformation Finite Element Models

During jack-up reinstallation, the footing undergoes progressive penetration and the soil is heavily remoulded during the process. Therefore, the numerical modelling of jack-up reinstallation response must allow for large deformation and strain dependent soil responses. Large deformation numerical modelling has been successfully applied by Hossain (2008) for the prediction of spudcan penetration responses in non-homogenous soil. Such large deformation numerical modelling method could be extended for the prediction of jack-up reinstallation response. However, there are a few key challenges in the development of a numerical model for this problem. Firstly, large deformation numerical modelling normally requires significant effort in coding and model development. Secondly, a full 3D numerical model is required to model the circular footing near a footprint in conical shape. Furthermore, the accuracy of such analysis heavily relies on the implementation of a realistic soil model with consolidation (during operation time of first jack-up installation and elapsed time before reinstallation). The modelling of large deformation in a full three-dimensional numerical model with advanced constitutive soil model requires significant computational time. However, once the model is established, parametric study of a wide range of variables could be conducted efficiently. The failure mechanisms throughout reinstallation could also be evaluated quantitatively and qualitatively. The consistent and high quality test results obtained in this study could be used for verification of the numerical model.

10.3.5 Development of Mitigation Methods

Mitigation methods currently adopted by the industry are stomping and infilling the footprint. These two methods only modify the footprint geometry and the shallow depth strength profile. They are, in principle, able to reduce the effect of footprint geometry and therefore mitigating the problem of relatively shallow depth reinstallation with long elapsed time. The effectiveness of infilling is also highly reliant on the properties of the infilling material. However, it is of major concern that the stark contrast in the strength of the infilling material and the soil around the footprint could increase footing loads, particularly a higher moment. Further experimental investigation on the effect of these mitigation measures to the shallow depth reinstallation is recommended. Furthermore, other mitigation methods are required for deep reinstallation near z_{FC}, and novel ideas could be considered.

There is increasing interest in the potential use of skirted footing as a mitigation measure for the jack-up reinstallation problem. However, the reinstallation test of a skirted footing near idealised footprint cavity in this study indicated that both the moment and horizontal forces were larger than for the flat-base footing. Although the reinstallation loadings are expected to reduce with the inclusion of jack-up structural properties in the model, a skirt’s effect on the jack-up movement is unknown. A more realistic assessment on the potential benefit of the use of skirt footing should be
conducted to consider both the reinstallation load and movement. This could be assessed by comparing the responses of the same flexible jack-up unit with the two different foundations.
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APPENDIX A. LITERATURE REVIEW OF THE VERTICAL CAPACITY OF A FOOTING ON A FLAT SURFACE

A.1 INTRODUCTION

In this appendix, an overview of the methods available for the determination of vertical capacity of shallow circular footing is first presented. Detailed discussion on two prediction methods: Houlsby and Martin (2003) and Hossain and Randolph (2009) developed specifically for spudcan penetration is then presented.

A.2 VERTICAL FORCE OF A CIRCULAR FOOTING ON A FLAT SURFACE

Traditionally, the bearing capacity of circular footings at different depths is approximated by applying a shape factor, \( s_c \), and a depth factor, \( d_c \), to Terzaghi’s (1943) bearing capacity equation. The modified bearing capacity equation takes the following form:

\[
q_u = s_c d_c N_c s_u D + \gamma' D
\]

(A.1)

Where \( B \) equals to the footing diameter and \( s_u \) is the undrained shear strength of soil. \( s_c \) and \( d_c \) proposed by various researchers are shown in Table A.1 and Table A.2. \( s_c \) was calculated based on the exact solution of \( N_c = 5.14 \) for strip footings. The equation for \( d_c \) applies to footings at different depths \( (z) \). However, the modified bearing capacity equation should be used with caution. Salgado et al. (2004) demonstrated that \( s_c \) and \( d_c \) are not independent. With the advancement of numerical modelling, \( s_c \) and \( d_c \) could be determined in combination rather than separately from Table A.1 and Table A.2. Examples of numerical modelling results shown by Salgado et al. (2004) and Gourvenec et al. (2006) are also presented in Table A.1.

Exact solutions are available for surface circular footings subjected to undrained conditions on weightless soil with constant \( s_u \). The bearing capacity factor, \( N_c \) for circular footings is affected by the base roughness. Eason and Shield (1960) presented an exact solution of \( N_c = 6.05 \) for a rough circular footing, and the footing failed with a Prandtl-type mechanism. The exact solution of \( N_c \) for a smooth footing was 5.69, as presented by Shield (1955). A Hill-type mechanism was suggested for circular smooth footings.
Appendix A: Literature Review of the Vertical Capacity of a Footing on a Flat Surface

Table A.1: Summary of the Shape Factors for Circular Footings Subjected to Undrained Loading

<table>
<thead>
<tr>
<th>Reference</th>
<th>Analysis Type</th>
<th>$s_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Skempton (1951)</td>
<td>Empirical</td>
<td>1.20</td>
</tr>
<tr>
<td>Meyerhof (1951)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hansen (1970)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vesic (1973)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Analysis</td>
<td>Rough=1.21</td>
</tr>
<tr>
<td>Gourvene et al. (2006)</td>
<td>Finite Element</td>
<td>Smooth = 1.09</td>
</tr>
<tr>
<td></td>
<td>Analysis</td>
<td>Rough = 1.16</td>
</tr>
</tbody>
</table>

Table A.2: Summary of Depth Factors for Footings Subjected to Undrained Loading

<table>
<thead>
<tr>
<th>Reference</th>
<th>$d_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Meyerhof (1951)</td>
<td>$d_c = 1 + 0.2 \frac{z}{B}$</td>
</tr>
<tr>
<td>Hansen (1970)</td>
<td>For $\frac{z}{B} \leq 1$, $d_c = 1 + 0.4 \frac{z}{B}$; For $\frac{z}{B} &gt; 1$, $d_c = 1 + 0.4 \tan^{-1} \frac{z}{B}$, where $\tan^{-1} \frac{z}{B}$ is in radians</td>
</tr>
<tr>
<td>Vesic (1973)</td>
<td>$d_c = 1 + 0.3 \tan^{-1} \frac{z}{B}$, where $\tan^{-1} \frac{z}{B}$ is in radians</td>
</tr>
<tr>
<td>ISO19901-4 (2003)</td>
<td>$d_c = 1 + 0.3 \tan^{-1} \frac{z}{B}$, where $\tan^{-1} \frac{z}{B}$ is in radians</td>
</tr>
</tbody>
</table>

For the prediction of the vertical capacity of a jack-up installation, the bearing capacity of a circular footing in non-homogeneous clay soil was more relevant than that of footing on soil with a constant $s_u$ (homogeneous soil). The $s_u$ profiles of non-homogeneous soil are often presented in the form of:

$$s_u = s_{um} + k z \quad (A.2)$$

For this type of material, a dimensionless strength factor $\kappa$ is often considered, and $\kappa$ can be computed based on:

$$\kappa = \frac{kD}{s_{um}} \quad (A.3)$$

Houlsby and Wroth (1983) listed $N_c$ for both smooth and rough circular footings for $0 \leq \kappa \leq 10$. Kusakabe et al. (1986) presented upper bound solutions for the problem. The Hill-type mechanism was observed in smooth footings and in rough footings with $\kappa > 1$. The change of failure mode with $\kappa$ and footing roughness is shown in Figure A.1.
Appendix A: Literature Review of the Vertical Capacity of a Footing on a Flat Surface

Tani and Craig (1995) considered both smooth and rough footings on soil with $0 \leq \kappa \leq 30$. Both $s_c$ and $d_c$ were computed, but the results for rough footings were found to be erroneous (Martin and Randolph, 2001). Exact solutions were presented by Martin (2001), and the change in failure mechanism with $\kappa$ and footing roughness was also reported. The Prandtl-type mechanism was only observed for rough footings on soil with $\kappa \leq 1$ or $2$.

A.3 VERTICAL FORCE OF SPUDCAN PENETRATION INTO FLAT SURFACE

There are more recent developments in the bearing capacity factors, especially for surface and embedded spudcan founding in non-homogeneous soil. Houlsby and Martin (2003) developed lower-bound solutions for conical footings in clay, assuming rigid-perfectly plastic soil behaviour with yielding governed by the Tresca criterion and an associated plastic flow rule. Tabulated values of $N_c$ are provided for the four variables of cone angle, footing base roughness, embedment depth and soil non-homogeneity.

Hossain (2008) developed a mechanism-based design approach based on a centrifuge model test and a finite element analysis. Hossain and Randolph (2009) extended the work by incorporating the strain-softening effect and the rate-dependency effect.

Both Houlsby and Martin (2003) and Hossain and Randolph (2009) adopted the same basic equations for the evaluation of the vertical force.

$$V = (N_c s_u + \gamma' z)A + \gamma' Vol$$  \hspace{1cm} (A.4)
where $N_c$ is the dimensionless bearing capacity factor and $s_u$ is the undrained shear strength. Both terms are referenced to the footing level $z$. $\gamma'$ is the effective unit weight of soil. $A$ and $Vol$ are the area and the volume of the footing, respectively. $s_u$ for flat-base footings and conical footings are defined in Figure A.2.

$Su = Sum + kz$

Figure A.2: Definition of Terminologies in Houlsby and Martin (2003) and Hossain and Randolph (2009)

Equation (A.4) is only valid for the case of an open cavity, that is, no soil collapse onto the footing. However, based on the observation from Hossain and Randolph (2009), soil started to collapse onto the footing and fill the spudcan cavity for penetration below the critical cavity depth, $H_s$. Hossain (2008) presented a method to compute $H_s$ through iteration. Based on Hossain (2010), it was assumed that backfilling occurs gradually between $H_s$ and 0.3 times the footing diameter (0.3D) below $H_s$. The vertical capacity during backfilling of the cavity could be predicted using:

$V = N_c s_u A + \gamma' z A - \left( \frac{\gamma' A - \gamma' Vol}{0.3D} \right) (z - H_s), \text{ For } H_s < z < H_s + 0.3D$  \hspace{1cm} (A.5)

and

$V = N_c s_u A + \gamma' Vol, \text{ For } z > H_s + 0.3D$  \hspace{1cm} (A.6)

Menzies and Roper (2008) compared these two methods with the conventional Skempton (1951) and Hansen (1970) methods. Menzies and Roper (2008) found that Houlsby and Martin (2003) and Hossain (2008) represented the lower and upper end estimation compared with the field records.

### A.4 REFERENCES


APPENDIX B. LITERATURE REVIEW OF THE VERTICAL CAPACITY OF A FOOTING NEAR SLOPING GROUND

B.1 INTRODUCTION

Unlike the case for jack-up installation into a flat surface, there is currently no guideline for the prediction of the vertical capacity of a jack-up installed near a footprint. Therefore, this appendix summaries the key literature on the prediction of bearing capacity of footings near sloping ground, with an emphasis placed on applications in undrained clay soil (the only soil type considered in this study).

The bearing capacity of footings near a slope is traditionally obtained by applying a slope correction factor, $g_c$, to Terzaghi’s (1943) general bearing capacity equation. In addition, there are other prediction methods adopting the limit analysis or limit equilibrium analysis technique. The key literatures are summarised in Table B.1. The definition of terminologies for the geometric parameters of slope and footings are defined in Figure B.1.

Most of the prediction methods were developed based on plane-strain conditions and are not directly applicable to slopes with distinct local loads (footings with finite $L/B$). Three-dimensional effect must be considered in such cases. The three-dimensional model is also more relevant for the study of the vertical capacity of a footing near a footprint cavity. Unfortunately, there are a relatively limited number of publications on the three-dimensional analyses of footings near sloping ground. The literatures on the three-dimensional analyses are presented in Table B.2.

![Figure B.1: Definition of Terminologies of Footings nearby a Slope](image)

Undrained Shear Strength: $s_u$ or $s_u + k_z$
Self-Weight: $\gamma'$
<table>
<thead>
<tr>
<th>Reference</th>
<th>Equation/Method</th>
<th>$s_u$</th>
<th>$\eta$</th>
<th>$z_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Meyerhof (1957)</td>
<td>$g_c$, Based on Stability Charts</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>Hansen (1970)</td>
<td>$g_c = 1 - \frac{20}{\pi + 2}$</td>
<td>×</td>
<td>√</td>
<td>×</td>
</tr>
<tr>
<td>Vesic (1975)</td>
<td>$N_c = 5.14 \left(1 - \frac{20}{\pi + 2}\right) \cdot \frac{\gamma B}{s_u} \sin \theta$</td>
<td>×</td>
<td>√</td>
<td>×</td>
</tr>
<tr>
<td>Bowles (1996)</td>
<td>Charts of $g_c$</td>
<td>×</td>
<td>✓</td>
<td>×</td>
</tr>
<tr>
<td>Drucker (1953)</td>
<td>Limit Analysis</td>
<td>×</td>
<td>$\eta = 30^\circ$</td>
<td>×</td>
</tr>
<tr>
<td>Davis and Booker (1973)</td>
<td>Limit Analysis</td>
<td>×</td>
<td>$\eta = 90^\circ$</td>
<td>×</td>
</tr>
<tr>
<td>Narita and Yamaguchi (1990)</td>
<td>Limit Equilibrium Method</td>
<td>$s_u/\gamma B = 0.5, 1, 5$</td>
<td>$\eta = 15^\circ, 35^\circ, 45^\circ, 60^\circ$</td>
<td>×</td>
</tr>
<tr>
<td>Kusakabe et al. (1981)</td>
<td>Limit Analysis</td>
<td>$s_u/\gamma B = 0.5, 1.0, 5$</td>
<td>$\eta = 15^\circ, 35^\circ, 40^\circ, 60^\circ$</td>
<td>✓ $z_s/B = 1.3$</td>
</tr>
<tr>
<td>Saran et al. (1989)</td>
<td>Limit Equilibrium Method and Limit Analysis</td>
<td>×</td>
<td>$\eta = 10^\circ - 50^\circ$ at $10^\circ$ intervals</td>
<td>×</td>
</tr>
<tr>
<td>Shiau et al. (2010)</td>
<td>Numerical Limit Analysis</td>
<td>$s_u/\gamma B = 0.5$</td>
<td>$\eta = 30^\circ, 60^\circ, 90^\circ$</td>
<td>✓</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$k_B/s_{um} = 1.23, 3.66, 5.03$</td>
<td>$\eta = 90^\circ (s_{um} = s_u + k_B)$</td>
<td>×</td>
</tr>
<tr>
<td>Georgiadis (2010a)</td>
<td>Finite Element Analysis</td>
<td>✓</td>
<td>✓</td>
<td>✓ $z_s/B = 0 - 26$</td>
</tr>
<tr>
<td>Georgiadis (2010b)</td>
<td>Limit Analysis</td>
<td>$s_u/\gamma B = 1.5, 2.5, 5.0$</td>
<td>$\eta = 15^\circ, 30^\circ, 45^\circ$</td>
<td>×</td>
</tr>
</tbody>
</table>

Table B.1: Summary of the Literature on the Two-Dimensional Vertical Bearing Capacity of Footings near a Slope in Undrained Clay Soil.
### Table B.2: Summary of the Literature on the Three-Dimensional Vertical Bearing Capacity of Footings on a Slope in Undrained Clay Soil

<table>
<thead>
<tr>
<th>Reference</th>
<th>Equation/Method</th>
<th>L/B</th>
<th>( \eta )</th>
<th>( \varphi ) (Stability Number)</th>
<th>( z_s/B )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Azzouz and Baligh (1982)</td>
<td>LEM</td>
<td>L/B = 1</td>
<td>9°</td>
<td>15°, 45°, 90°</td>
<td>z_s/B = 4, 2.1</td>
</tr>
<tr>
<td>Michalowski (1989)</td>
<td>Limit Analysis</td>
<td>L/B = 1.2</td>
<td>9°</td>
<td>45°</td>
<td>z_s/B = 3</td>
</tr>
<tr>
<td>Wei et al. (2009)</td>
<td>Numerical Analyses</td>
<td>L/B = 0.10</td>
<td>9°</td>
<td>45°</td>
<td>z_s/B = 2.5</td>
</tr>
</tbody>
</table>
The literature review presented above provided useful information for understanding the influential parameters in terms of footing capacity. However, none of these methods could be used directly for the prediction of the vertical capacity of jack-up reinstallation problems. A prediction method needs to be developed to incorporate the following unique features of the problem:

1) slope in non-homogeneous soil;
2) circular footing; and
3) conical slope with a finite slope length $l$.

### B.2 REFERENCES


APPENDIX C. PREDICTION OF $z_{\text{CONTACT}}$

C.1 INTRODUCTION

The ground profiles during the half-footing reinstallation tests were digitally measured from the photos using GeoPIV8 programme developed by White et al. (2003). The change in the ground profile with penetration depth for test TB-10D-HF is shown in Figure C.1. Heaving was observed immediately adjacent to the footing. However, the volume of soil heaving was negligible compared with the volume of the soil bund building up at the footprint toe. Because the footing was experiencing undrained loading conditions, there was no change in the soil volume. It could therefore be assumed that the soil displaced by the footing is equal to the volume of the soil bund at the footprint toe.

Figure C.1: Change of Ground Profile during Half-Footing Reinstallation Test TB-10D-HF
In addition to the two-dimensional view obtained from the half-footing tests, plan view pictures were also captured during the full-footing reinstallation tests. This allows a qualitative investigation of soil movement in three dimensions. While heaving occurred around the footing, the build up of the soil bund was concentrated near the slope toe. The volume of the soil bund reduced radially with distance away from the footing centre.

For the prediction of \( z_{\text{contact}} \), the ground profile could be simplified, as shown in Figure C.3. The length (into the page for the section view) of the soil bund was assumed to be the same as the footing diameter. With the known parameters \( z_F \), \( D \) and \( \beta \), the unknown \( z_{\text{contact}} \) can be solved by equating \( \text{Vol}_{\text{bund}} \) to \( \text{Vol}_{\text{Foot}} \). Therefore, \( z_{\text{contact}} \) can be predicted by solving a simple trigonometry problem.
Figure C.2: Plan View of Full-Footing Reinstallation Test TB-2D-05D
This prediction method was verified by comparing the output with the measured $z_{contact}$ from half-footing reinstallation tests. The comparison shown in Table C-1 indicates that the predicted $z_{contact}$ was shallower than the measured data. While the predicted $z_{contact}$ for TB-10D-HF was 27% lower than the measured values, the prediction for TC-12D-HF was three times lower than the measured value. The under-estimation was likely caused by neglecting the soil heaving and the radial change of Vol bund. Measurement error could be another source of the discrepancy. The $z_{contact}$ measured from the half-footing tests was recorded in mm, and a 27% difference between the measured and
predicted $z_{\text{contact}}$ corresponds to a 1.63mm difference only. The prediction agreed with the general trend of increasing $z_{\text{contact}}$ with decreasing $\beta$ or increasing footprint size (from TB and TC). Although this method under-estimated $z_{\text{contact}}$, especially for footprint TC, it provided a simple means of initial estimation. The method could be enhanced by considering the more complex wedge shaped soil movement and the heaving effect. However, this is beyond the scope of this study.

**Table C-1: Comparison of $z_{\text{contact}}$ from the Half-Footing Test and from the Prediction Method**

<table>
<thead>
<tr>
<th>Half-Footing Test</th>
<th>Measured $z_{\text{contact}}$ (mm)</th>
<th>Predicted $z_{\text{contact}}$ (mm)</th>
<th>Measured $z_{\text{contact}}/D$</th>
<th>Predicted $z_{\text{contact}}/D$</th>
</tr>
</thead>
<tbody>
<tr>
<td>TB-05D-HF</td>
<td>9.6</td>
<td>5.2</td>
<td>0.16</td>
<td>0.09</td>
</tr>
<tr>
<td>TB-10D-HF</td>
<td>6.6</td>
<td>5.0</td>
<td>0.11</td>
<td>0.08</td>
</tr>
<tr>
<td>TC-12D-HF</td>
<td>6.0</td>
<td>1.5</td>
<td>0.10</td>
<td>0.03</td>
</tr>
</tbody>
</table>

**C.2 REFERENCES**

APPENDIX D. CALCULATION OF THE REDUCTION FACTOR USING 2D-FE

D.1 INTRODUCTION

The purpose of this appendix is to present the development of reduction factor at ground level, $R_{Fi}$, using two-dimensional numerical (2D-FE) analyses. The major advantage of 2D-FE analysis is its high computation efficiency. Therefore, a parametric study could be conducted efficiently to investigate the effect of each geometric parameter on the vertical capacity. 2D-FE analyses were conducted to investigate the change of $R_{Fi}$ with footprint geometries and offset distance during jack-up reinstallation. This provides useful information for subsequent calculation of $R_{Fi}$ using three-dimensional numerical analyses (2D-FE).

The definition of terminologies for the geometric parameters of 2D-FE models are defined in Figure B.1. The jack-up reinstallation problem was simplified as a strip footing located near an infinite slope (infinite $l$) for analysis of the plane-strain condition. The 2D-FE analyses were performed using the commercial finite element package PLAXIS (Brinkgreve et al. 2008), which was specifically developed for the analysis of deformation and stability in geotechnical engineering problems. The latest PLAXIS 2D version 9.0 was used in the analyses.

D.2 2D-FE MODEL

The plane-strain soil domain was chosen to be 9D in width and 4D in depth. The soil domain was fully fixed from movement and rotation at the bottom boundary and was free to slide in the vertical direction along the vertical boundaries. The mesh used for the analyses of footings on flat surfaces is shown in Figure D.1. This mesh contains 4000 elements. The mesh was constructed with a more refined mesh near the critical region (the footing). This allowed the relatively large domain to be solved efficiently without sacrificing the accuracy.
Appendix D: Calculation of the Reduction Factor Using 2D-FE

The soil was modelled as an elasto-plastic material obeying a Tresca yield criterion to simulate the undrained penetration of footings into clay soil. A stiffness ratio of 500 was adopted, which was within the range common for soft clays. The stiffness has a negligible effect because soil deformation is irrelevant to the bearing capacity calculation, and only the ultimate loads were adopted. An effective weight was assumed, and geostatic stress was implemented by letting $K_o=1$. The basic modelling assumptions and soil parameters are shown in Table D.1.

Table D.1: Soil Model and Parameters Adopted in the 2D-FE Model

<table>
<thead>
<tr>
<th>Soil Model</th>
<th>Undrained Tresca</th>
</tr>
</thead>
<tbody>
<tr>
<td>Undrained Shear Strength, $s_u$</td>
<td>(1) Homogeneous Clay Soil, $s_u = 15kPa,$</td>
</tr>
<tr>
<td></td>
<td>(2) Non-Homogeneous Clay Soil, $s_u = 5+1.68z$</td>
</tr>
<tr>
<td>Effective Unit Weight, $\gamma'$</td>
<td>7.65 kN/m$^3$</td>
</tr>
<tr>
<td>Young's Modulus, $E$ ($E = 500s_u$)</td>
<td>(1) Homogeneous Clay Soil, $E = 7500kPa,$</td>
</tr>
<tr>
<td></td>
<td>(2) Non-Homogeneous Clay Soil, $E = 2500+840z$ kPa</td>
</tr>
<tr>
<td>Poisson Ratio, $\nu'$</td>
<td>0.49</td>
</tr>
</tbody>
</table>
Appendix D: Calculation of the Reduction Factor Using 2D-FE

The main focus of this numerical study was to determine the bearing capacity (in terms of reduction factor RF_i) of footings near footprint cavities. The footing model was 15m wide and fully fixed from sliding (in the horizontal direction) and from rotation at the footing reference level. A smooth interface was modelled between the footing and the soil. The footing load was prescribed by vertical displacement over the loading width to enable post-failure conditions to be observed.

To investigate the performance of the mesh and the soil model, two sets of control models were simulated for comparison with analytical solutions. In the first set of control models, 3 models with value of x_contact of 1.5m, 7.5m and 15m on a flat surface in homogenous clay (constant undrained shear strength, s_u) were considered. In the second set of control tests, the same three footings were considered, but in non-homogeneous clay (s_u increases linearly with depth). The undrained soil strength profile in the non-homogeneous model was the same as the one in the soil sample adopted for the full-footing reinstallation tests (refer to Chapter 3 for details).

The load-settlement curve and failure mechanisms of the 15m wide footing in the homogeneous soil model are presented in Figure D.2. Table D.2 shows that the N_c values calculated from 2D-FE was 5.27, which was only 3% higher than the exact solution of 5.14 obtained by Prandtl (1920). The soil failed via the Prandtl-type mechanism, which is typical for footings in homogeneous clay soil. Figure D.3 shows another 2D-FE result for a 15m wide footing in the non-homogeneous soil model. Table D.3 shows that the N_c was 8.47, which was less than 1% higher than the exact solution of 8.43 obtained by Davis and Booker (1973). A Hill-type mechanism was observed from the 2D-FE analysis, which agreed with the failure mechanism suggested by Davis and Booker (1973) for smooth footings on non-homogeneous soil.

Table D.2: Comparison of 2D-FE Output with Analytical Solutions (Homogeneous Soil)

<table>
<thead>
<tr>
<th>B (m)</th>
<th>Corresponding β</th>
<th>N_c (Prandtl, 1920)</th>
<th>N_c (2D-FE)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>0.5D</td>
<td>5.14</td>
<td>5.27</td>
</tr>
<tr>
<td>7.5</td>
<td>1.0D</td>
<td></td>
<td>5.20</td>
</tr>
<tr>
<td>15</td>
<td>1.5D</td>
<td></td>
<td>5.17</td>
</tr>
</tbody>
</table>

Figure D.2: 2D-FE Output for a Smooth Strip Footing on Homogenous Clay Soil
Table D.3: Comparison of 2D-FE Output with Analytical Solutions (Non-Homogeneous Soil)

<table>
<thead>
<tr>
<th>B (m)</th>
<th>Corresponding $\beta$</th>
<th>$N_c$ (Davis and Booker, 1973)</th>
<th>$N_c$ (2D-FE)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>0.5D</td>
<td>5.69</td>
<td>5.75</td>
</tr>
<tr>
<td>7.5</td>
<td>1.0D</td>
<td>7.10</td>
<td>7.10</td>
</tr>
<tr>
<td>15</td>
<td>1.5D</td>
<td>8.45</td>
<td>8.43</td>
</tr>
</tbody>
</table>

The mesh was then modified for the analysis of footings near footprint cavities (Figure D.4 and Figure D.5). To model footings located at $\beta<1.5D$ from the footprint centre, the loading was applied over a loading width of $x_{\text{contact}}$ rather than B (Figure D.4). To address the slight variation of $N_c$ resulting from the three different models, $R_F$ was determined by comparing the reduced bearing capacity factor ($N_{RF}$) with the $N_c$ (of the flat surface model) obtained from the same mesh model.
The reduced bearing capacity factor $N_{RF}$ for footings near footprint cavities in homogeneous soil is shown in Table D.4. The reduction factor at the touchdown level, $RF_i$, was determined as the ratio of $N_{RF}$ to $N_c$ for a flat footing surface. For the case of $\beta=0.5D$, the theoretical $N_c$ at the touchdown level should be zero because the footing was not in contact with the soil. However, as revealed from the half-footing tests, the soil moved rapidly after footing touchdown, and the footing should soon gain some vertical resistance. This was validated by the non-zero vertical forces measured during the full-footing reinstallation tests. This rapid gain in the vertical capacity was modelled by assuming an offset distance of 0.1D ($\beta=1.0$, $x_{contact}=1.5m$) at the touchdown level. Although the actual offset distance between the centre of the footprint to the centre of the footing became 0.6D instead of 0.5D, the analysis still applies to the case of 0.5D.
D.2.1 Reduction Factor for the 2D Jack-up Reinstallation Model in Homogeneous Soil

A homogeneous soil model was first analysed to gain initial understanding of the geometric effect of the footprint. The failure mechanisms and $N_c$ were also compared with the findings from other publications. A constant undrained shear strength, $s_u$, of 15kPa was adopted, giving the homogeneous soil model sufficient strength to avoid slope failure of footprint cavities under gravity.

The reduced bearing capacity factors ($N_{RF}$) and the reduction factors $RF_i$ are summarised in Table D.4 and Figure D.6. $RF_i$ increased linearly with offset distance $\beta$ and increased when the footprint type changed from the steepest and deepest footprint TC to the flattest and shallowest footprint TA. At $\beta=0.5D$, $RF_i$ was relatively insensitive to the change of footprint type, and it decreased by 30% when changing the footprint from type TA to TC. However, when $\beta=1.5D$ was considered, $RF_i$ reduced by more than three times when changing from footprint type TA to TC.

Table D.4: Reduced Bearing Capacity Factor and Reduction Factor from 2D-FE Analyses of Strip Footings Near a Slope in Homogeneous Soil ($s_u=15$kPa)

<table>
<thead>
<tr>
<th>$x_{contact}$ (m)</th>
<th>$\beta$</th>
<th>$N_c$ (FS)</th>
<th>$N_{RF}$</th>
<th>$RF_i$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>TA</td>
<td>TB</td>
<td>TC</td>
</tr>
<tr>
<td>1.5</td>
<td>0.5D</td>
<td>5.27</td>
<td>0.51</td>
<td>0.46</td>
</tr>
<tr>
<td>7.5</td>
<td>1.0D</td>
<td>5.20</td>
<td>2.24</td>
<td>1.73</td>
</tr>
<tr>
<td>15</td>
<td>1.5D</td>
<td>5.17</td>
<td>4.18</td>
<td>3.12</td>
</tr>
</tbody>
</table>

Figure D.6: Reduction Factor from 2D-FE Analyses of Strip Footings Near a Slope in Homogeneous Soil ($s_u=15$kPa)

For footing reinstallation at $\beta=1.5D$, the footing contact area was the same as the full footing area; therefore, $N_{RF}$ was the same as $N_c$. $N_c$ values obtained from 2D-FE were compared with the theoretical $N_c$ presented in Hansen (1970), Bowles (1996) and
Georgiadis (2010). Table D.5 shows that both Hansen (1970) and Bowles (1996) significantly over-predicted $N_c$, and this was because the self-weight $\gamma$ and the slope height $z_s$ were ignored in these two studies. Georgiadis (2010a) considered a minimum $s_u/\gamma B$ of 0.5, which was significantly larger than the $s_u/\gamma B$ of 0.13 considered in this study. However, Georgiadis (2010a) still provided the best $N_c$ approximation, with a 30% difference. The comparison demonstrated the importance of incorporating $\gamma$ and the $z_s$ for the accurate prediction of $N_c$, particularly for cases involving relatively steep slopes.

Table D.5: Comparison of the Bearing Capacity Factor from 2D-FE Analyses of Strip Footings Near a Slope in Homogeneous Soil ($s_u$=15kPa) with Theoretical Values

<table>
<thead>
<tr>
<th>Footprint Model</th>
<th>$N_c$ from 2D-FE</th>
<th>Theoretical $N_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$T_A$</td>
<td>$T_B$</td>
</tr>
<tr>
<td>$s_u$-T-A-15D</td>
<td>4.18</td>
<td>4.81</td>
</tr>
<tr>
<td>$s_u$-T-B-15D</td>
<td>3.12</td>
<td>4.50</td>
</tr>
<tr>
<td>$s_u$-T-C-15D</td>
<td>0.91</td>
<td>3.96</td>
</tr>
</tbody>
</table>

The failure mechanisms governing the change of $RF_i$ were explored with reference to the deformed model mesh and incremental shear strain contours that are presented in Figure D.7 to Figure D.9. As $\beta$ increased, more soil was mobilised under the footing, and the failure slip intercepted the footprint toe. $RF_i$ in these cases was therefore influenced by the footprint toe level ($z_F$). The failure mechanisms changed from the Prandtl-type mechanisms in TA and TB to predominantly slope failure in TC. This change in the failure mechanism would contribute to the reduction in $RF_i$. The observation of the change in the failure mechanisms was in good agreement with Georgiadis (2010). The failure mechanisms suggested by Georgiadis (2010) is shown in Figure D.10. As the footprint size increased from TA to TB, the $z_s/B$ ratio increased 4 times. The failure mechanism therefore changed from Mechanism 1 (half-Prandtl-type failure) to Mechanism 3 (overall slope failure). For $\beta$=0.5D cases, the failure regions were the smallest compared with cases with larger $\beta$ and the failure region confined near the slope crest.
Appendix D: Calculation of the Reduction Factor Using 2D-FE

Figure D.7: Deformed Meshes and Incremental Shear Strain Contours for Footprint TA Cases ($s_u = 15$kPa$)

Figure D.8: Deformed Meshes and Incremental Shear Strain Contours for Footprint TB Cases ($s_u = 15$kPa$)
Figure D.9: Deformed Meshes and Incremental Shear Strain Contours for Footprint TC Cases (s_u=15kPa)

Mechanism 1
Prandtl-type with the Slip Surface Intercepting the Slope Toe

Mechanism 2
Prandtl-type with the Slip Surface Confined within the Slope Face

Mechanism 3
Overall Slope Failure

Figure D.10: Three Different Failure Mechanisms of Footings on a Slope (Georgiadis 2010)
Further parametric study was conducted to investigate the effect of \( s_u \) on \( R_F_i \). Figure D.11 shows that \( R_F_i \) increased with \( s_u \) in a non-linear manner. \( R_F_i \) increased by 2.8 times as \( s_u \) doubled. Therefore, \( R_F_i \) is a function of \( \beta \), footprint type (\( \eta \) and \( z_s \)) and \( s_u \).

![Figure D.11: Change in RF_i with Undrained Shear Strength](image)

### D.2.2 Reduction Factor for the 2D Jack-up Reinstallation Model in Non-Homogenous Soil

For a direct comparison between the 2D-FE analyses output and the full-footing reinstallation tests (details presented in Chapter 3), the \( s_u \) profile adopted in the 2D-FE analyses of the non-homogeneous soil model was the same as the one used in the full-footing reinstallation test.

\( N_{RF} \) and \( R_F_i \) for footings near footprint cavities in non-homogeneous soil are shown in Table D.6 and Figure D.12. \( R_F_i \) increased almost linearly with \( \beta \). However, the effect of footprint type on \( R_F_i \) was less vigorous compared with the case of a homogeneous slope. For \( \beta=1.5 \), \( R_F_i \) only increased by 27% when changing the footprint type from TA to TC.

<table>
<thead>
<tr>
<th>( x_{contact} ) (m)</th>
<th>( \beta )</th>
<th>( N_c ) (FS)</th>
<th>( N_{RF} )</th>
<th>( R_F_i )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>TA</td>
<td>TB</td>
</tr>
<tr>
<td>1.5</td>
<td>0.5D</td>
<td>5.75</td>
<td>0.56</td>
<td>0.52</td>
</tr>
<tr>
<td>7.5</td>
<td>1.0D</td>
<td>7.10</td>
<td>3.36</td>
<td>3.15</td>
</tr>
<tr>
<td>15</td>
<td>1.5D</td>
<td>8.43</td>
<td>8.04</td>
<td>7.51</td>
</tr>
</tbody>
</table>

Table D.6: Reduced Bearing Capacity Factor and Reduction Factor from 2D FE Analyses of Strip Footings Near a Slope in Non-homogeneous Soil (kD/\( s_{um}=5 \))
The failure mechanisms are presented as deformed meshes and incremental shear strain contours in Figure D.13 to Figure D.15. A different failure mechanism was observed compared with that observed for homogeneous soil. A skewed Hill-type mechanism was observed for footings near footprint cavity TA and TB in non-homogeneous soil. The bearing failure mechanisms were contained within the face of the slope. The midpoints of the Hill-type mechanisms were offset from the centre of the footing. For footprint cavity TC, the mid-point of the two-way failure mechanisms shifted to the footing edge and became almost a half Hill-type mechanism. Although the failure slip also intercepted the slope toe, a radial wedge and a passive wedge could still be observed in TC-15D, and the slope still failed via a Hill-type mechanism.
Further investigation of the failure mechanisms was made by comparing the 2D-FE analyses results with the half-footing reinstallation test. Figure D.16 and Figure D.17 show that the soil movement obtained from the two methods were comparable. The shifting of the mid-point of the Hill-type mechanisms and the increasing size of the...
Appendix D: Calculation of the Reduction Factor Using 2D-FE

...failure region when the footprint changed from TB to TC were observed in both 2D-FE analyses and the test results. Therefore, simplifying the jack-up reinstallation problem for the 2D-FE modelling in plane-strain conditions provided useful information for the investigation of the failure mechanisms of circular footing located near footprint cavities.

![Figure D.16: Comparison of Soil Movement from 2D-FE Analyses with the Half-Footing Reinstallation Test (Footprint TB)](image)

![Figure D.17: Comparison of Soil Movement from 2D-FE Analyses with the Half-Footing Reinstallation Test (Footprint TC)](image)

D.3 CONCLUDING REMARKS

2D-FE analyses were undertaken to investigate the change of vertical capacity with different soil strengths, footprint geometries and offset distances. The inclusion of the self-weight of the soil and the slope height were critical to the accurate formulation of $N_c$. $RF_i$ was a function of the slope angle, slope height, offset distance, footing width and soil strength. While the Prandtl-type mechanism and the slope failure mechanism were observed in homogeneous soil, a skewed Hill-type mechanism was observed for a slope in non-homogeneous soil.

D.4 REFERENCES


APPENDIX E. CALCULATION OF THE REDUCTION FACTOR USING 3D-FE

E.1 INTRODUCTION

The purpose of this appendix is to present the reduction factor at ground level, RF₁ for the prediction of vertical capacity of footing during reinstallation. RF₁ was determined from three-dimensional numerical (3D-FE) analyses. The results from 3D-FE analyses were also compared with the 2D-FE results to investigate the three-dimensional effect of the problem.

E.2 3D-FE MODEL

3D-FE was conducted using the commercial finite element software ABAQUS (Dassault-Systemes 2009). The finite element model used for analysis of a circular footing of 15m in diameter (D=15m) near an idealised footprint cavity with a conical shape is shown in Figure E.1. The geometry and loading condition of the jack-up reinstallation problem was symmetrical about the vertical plane of symmetry; therefore, the mesh only modelled a half-footing and half-soil domain to improve the computation efficiency. The domain extended 3 times the footing diameter from the edge of the footing and 3.33D beneath the footing. The symmetrical condition was modelled by prescribing zero-displacement boundary conditions to prevent out-of-plane displacements at the plane of symmetry. The outer boundaries were free to slide vertically. The base of the mesh is fixed in all three co-ordinate directions. The boundary conditions are shown in Figure E.1.

The mesh was constructed with a more refined mesh near the footing and within the footprint to achieve a time-efficient model without compromising accuracy. Three sets of meshes were created for modelling footings at three different offset distances from the footprint centre (β=0.5D, 1.0D and 1.5D). The models adopted in the modelling problem with footprint TB at three different β are shown in Figure E.2 to Figure E.4. Each model comprised approximately 40,000 first-order fully integrated hexahedral hybrid elements. These models were modified locally within the footprint area to consider footings on flat surface, near footprint TA, TB or TC.
Appendix E: Calculation of the Reduction Factor Using 3D-FE

Figure E.1: The Domain and Boundary Conditions of the Three-Dimensional Numerical Model

Figure E.2: 3D FEM Model for a Footing at an Offset Distance of 0.5D from Footprint Type B (Model TB-05D)
Appendix E: Calculation of the Reduction Factor Using 3D-FE

Figure E.3: 3D FEM Model for a Footing at an Offset Distance of 1.0D from Footprint Type B (Model TB-10D)

Figure E.4: 3D FEM Model for a Footing at an Offset Distance of 1.5D from Footprint Type B (Model TB-15D)
The soil model and the material properties adopted in the 3D-FE model are identical to the 2D-FE model. The details are listed in Table D.1. A 15m diameter footing was represented as a discrete rigid body, with loads and displacements related to a single load reference point (LRP). The footing load was prescribed by vertical displacement at the reference point, which was taken as the midpoint along the base of the footing. The footing was restrained to movement and rotation in all the other directions. The analyses considered a smooth interface between the footing and the soil, simulating a smooth-based footing.

The accuracy of the three sets of models was first verified by considering a model of a circular footing on a flat surface. The meshes presented in Figure E.2 to Figure E.4 were modified with a slope angle of zero (i.e., a flat surface). The analysis output for a smooth, circular footing on a flat surface in homogeneous soil is summarised in Figure E.1. The $N_c$ values obtained from the 3D-FE analyses ranged from 5.50 to 5.51 and were 3.5% smaller than the exact solution presented in Shield (1955). Table E.2 shows the results for a smooth, circular footing on a flat surface in non-homogeneous soil. There was less than 1% difference between the 3D-FE analysis output and the exact solution from Houlsby and Martin (2003). The slight under-prediction of 3D-FE analyses was also reported in Gourvenec and Randolph (2003). This trend is likely related to the larger distortion of the elements local to the footing.

Table E.1: Bearing Capacity Factor of a Smooth, Circular Footing on a Flat Surface (Homogeneous Soil with $s_u=15kPa$

<table>
<thead>
<tr>
<th>Model Name</th>
<th>Shield (1955)</th>
<th>$N_c$ (3D-FE)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Su-FS-05D</td>
<td>5.69</td>
<td>5.50</td>
</tr>
<tr>
<td>Su-FS-10D</td>
<td></td>
<td>5.52</td>
</tr>
<tr>
<td>Su-FS-15D</td>
<td></td>
<td>5.51</td>
</tr>
</tbody>
</table>

Table E.2: Bearing Capacity Factor of a Smooth, Circular Footing on a Flat Surface (Non-Homogeneous Soil with $kD/s_{um}=5$)

<table>
<thead>
<tr>
<th>Model Name</th>
<th>Houlsby and Martin (2003)</th>
<th>$N_c$ (3D-FE)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Suk-FS-05D</td>
<td>7.94</td>
<td>7.85</td>
</tr>
<tr>
<td>Suk-FS-10D</td>
<td></td>
<td>7.94</td>
</tr>
<tr>
<td>Suk-FS-15D</td>
<td></td>
<td>7.93</td>
</tr>
</tbody>
</table>

Failure mechanisms from model Su-FS-10D and Suk-FS-10D are presented in Figure E.5 and Figure E.6. Hill-type mechanisms were observed in both cases, which matched with the upper bound failure mechanisms suggested by Shield (1955) and Houlsby and Martin (2003). The failure region of the footing on non-homogeneous soil was
shallower and more confined around the footing perimeter when compared with the homogeneous soil model.

Figure E.5: 3D-FE Analysis Output for a Smooth, Circular Footing on Homogenous Clay Soil with $s_u=15\text{kPa}$ (Model Su-FS-10D)

![Graph showing Normalised Displacement, $v/D$ against $N_c$]

$N_c=5.52$

Figure E.6: 3D-FE Analysis Output for a Smooth, Circular Footing on Non-Homogenous Clay Soil with $kD/s_{um}=5$ (Model Suk-FS-10D)

![Graph showing Normalised Displacement, $v/D$ against $N_c$]

$N_c=7.94$

### E.2.1 Reduction Factor for 3D Jack-up Reinstallation in Homogenous Soil

Table E.3 summarises $N_{RF}$ and $RF_i$ from the 3D-FE analyses of footings near footprint cavities in homogenous soils. The $RF_i$ and $N_{RF}$ resulting from footings at one offset distance were compared with $N_c$ obtained from the corresponding flat surface model (i.e., TA-05D vs. FS-05D) to reflect the varying error of different meshes. The changes of $RF_i$ with $\beta$ and footprint types are shown in Table E.3. $RF_i$ increased approximately linearly with offset distance, which was similar to the observation from the 2D-FE analysis output. However, the change of $RF_i$ with footprint type was relatively small, indicating that a circular footing near a circular footprint cavity was less sensitive to the change in the footprint geometry compared with the strip footing on a plane-strain slope.
Table E.3: The Reduced Bearing Capacity Factor and the Reduction Factor from 3D-FE Analyses of Circular Footings Near Slopes in Homogeneous Soil (\(s_u=15\text{kPa}\))

<table>
<thead>
<tr>
<th>(\beta)</th>
<th>(N_c) (FS)</th>
<th>(N_{RF})</th>
<th>(RF_i)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TA</td>
<td>TB</td>
<td>TC</td>
<td>TA</td>
</tr>
<tr>
<td>0.5D</td>
<td>5.50</td>
<td>0.36</td>
<td>0.34</td>
</tr>
<tr>
<td>1.0D</td>
<td>5.52</td>
<td>2.83</td>
<td>2.59</td>
</tr>
<tr>
<td>1.5D</td>
<td>5.51</td>
<td>5.40</td>
<td>5.16</td>
</tr>
</tbody>
</table>

Figure E.7: Reduction Factor from 3D-FE Analyses of Circular Footings Near Slopes in Homogeneous Soil (\(s_u=15\text{kPa}\))

The 3D-FE analysis output was compared with the displacement contour and incremental shear strain contour shown in Figure E.8 to Figure E.10. Skewed Hill-type mechanisms were observed in footings near footprint TA and TB. Although this is different from the Prandtl-type mechanisms observed from the 2D-FE analyses, this agreed with the Hill-type mechanisms suggested for smooth, circular footings on a flat surface. The difference in failure mechanisms explained the difference in vertical capacity predicted by the 2D-FE and 3D-FE analyses.

When footprint type changed to the steepest and deepest TC footprint, the failure mechanism changed to the Prandtl-type mechanism with the slip plane intercepting the footprint toe, rather than the slope failure observed in the 2D-FE analysis. A possible reason was that the plane-strain slope TC has a marginal factor of safety (\(F=1.09\)). Therefore, only a relatively small vertical load in the strip footing could trigger slope failure. However, for the circular footprint cavity TC, the three-dimensional slope failure slip was in a wedge shape, and the side edges of the slip could provide additional shear resistance to the footing load. The \(F\) of a three-dimensional slope was estimated from Gens et al. (1988). The \(F\) of the slope increased to 1.70 when three-dimensional effects were considered. This indicated that the three-dimensional slope could sustain a
larger vertical load and therefore have a higher $N_c$ when compared with the 2D plain-strain condition.

Figure E.8: Displacement Contour and Incremental Shear Strain Contours for Footprint TA Cases ($s_u=15\text{kPa}$)
Figure E.9: Displacement Contour and Incremental Shear Strain Contours for Footprint TB Cases ($s_u=15\text{kPa}$)
The three-dimensional effects were further quantified by considering the ratio between the $N_{RF}\ (3D\text{-FE})$ and $N_{RF}\ (2D\text{-FE})$. This ratio was called $F_{3D}$ and was defined as:

$$F_{3D} = \frac{N_{RF}\ (3D)}{N_{RF}\ (2D)}$$

Figure E.11 shows that the $F_{3D}$ values for reinstallation near footprints TA and TB were approximately 1.0. This indicated that the reinstallation responses were relatively insensitive to the three-dimensional effect. However, for footprint TC, which was the steepest and deepest footprint, the $RF_i$ from the 2D-FE analyses was 4 times smaller than that from the 3D-FE analyses. The 2D-FE analyses significantly under-estimated the vertical bearing capacity of footings near steep footprints.
Appendix E: Calculation of the Reduction Factor Using 3D-FE

Figure E.11: Three-Dimensional Factor for Footings at Different Offset Distances from Different Footprint Types (Homogeneous Soil with $su=15\text{kPa}$)

E.2.2 Reduction Factor for 3D Jack-up Reinstallation in Non-Homogenous Soil

Table E.4 summarised $N_{RF}$ and $RF_i$ from the 3D-FE analysis of footings near footprint cavities in non-homogenous soils. Figure E.12 shows that the $RF_i$ increased linearly with $\beta$ and footprint types. $RF_i$ varied with $\beta$ but was relatively insensitive to the change in footprint type.

Table E.4: Reduced Bearing Capacity Factor and Reduction Factor from 3D-FE Analyses of Circular Footings Near Slopes in Non-Homogeneous Soil ($kD/s_{u}=5$)

<table>
<thead>
<tr>
<th>$\beta$</th>
<th>$N_c$ (FS)</th>
<th>$N_{RF}$</th>
<th>$RF_i$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>TA</td>
<td>TB</td>
</tr>
<tr>
<td>0.5D</td>
<td>7.85</td>
<td>0.43</td>
<td>0.40</td>
</tr>
<tr>
<td>1.0D</td>
<td>7.94</td>
<td>3.83</td>
<td>3.64</td>
</tr>
<tr>
<td>1.5D</td>
<td>7.93</td>
<td>7.83</td>
<td>7.72</td>
</tr>
</tbody>
</table>
The failure mechanisms of footings on footprint cavities in non-homogeneous soil are summarised in Figure E.13 to Figure E.15. The skewed Hill-type mechanism was observed in all of the cases. With soil strength increasing with depth, the failure regions were shallower and more concentrated around the footing perimeter when compared with the case of footings on soil with constant $s_u$. The “skewness” of the mechanism increased when changing the footprint type from TA to the steeper and deeper footprint TB and TC. Eccentricity was also computed from the analyses output for the development of a moment prediction method.
Figure E.13: Displacement Contour and Incremental Shear Strain Contours for Footprint TA Cases ($kD/s_{un}=5$kPa)
Figure E.14: Displacement Contour and Incremental Shear Strain Contours for Footprint TB Cases (kD/s_{num}=5kPa)
Appendix E: Calculation of the Reduction Factor Using 3D-FE

The boundary conditions of the 3D-FE model with a half-soil domain and a half-footing model were essentially identical to the half-footing reinstallation tests (the details are presented in Chapter 4). The soil movement from the 3D-FE model and the experimental investigation are compared in Figure E.16 and Figure E.17. Although the soil movement during the half-footing reinstallation test was slightly deeper than the one from the 3D-FE analyses, the same skewed Hill-type mechanism were observed from both investigations. 3D-FE analysis is therefore a useful tool for the investigation of shallow depth failure mechanisms during jack-up reinstallation near footprints.
The three-dimensional factor, $F_{3D}$, of footings reinstalled in non-homogeneous soil is shown in Figure E.18. $F_{3D}$ gradually increased from 0.5 to a maximum of 1.34 with $\beta$ increasing from 0.5D to 1.5D. This indicated that 2D-FE analysis over-estimated the RFi at $\beta=0.5D$ but under-estimated it for cases with $\beta>1.0D$. Similar to the observation from footings in homogeneous soil, the steepest and deepest footprint TC was most sensitive to the three-dimensional effect. However, $F_{3D}$ was within the range of 0.5 to 1.34, which was smaller than the case of homogeneous soil. For reinstallation at $\beta>1.0$ from footprint TA and TB, RFi were insensitive to the three-dimension effect. The maximum $F_{3D}$ was less than 1.09, indicating only a 10% difference between the 2D-FE and 3D-FE analysis output.
Appendix E: Calculation of the Reduction Factor Using 3D-FE

Figure E.18: Three-Dimensional Factor for Footings at Different Offset Distances from Different Footprint Types (Non-Homogeneous Soil (kD/s_{num}=5))

E.3 CONCLUDING REMARKS

The comparison between 2D-FE and 3D-FE analyses indicated that 2D-FE analysis provided useful initial estimation of the failure mechanisms and the RF for the jack-up reinstallation problem. The skewed Hill-type mechanisms observed from the 2D-FE and 3D-FE analysis outputs were comparable to the half-footing reinstallation test results. The RF values obtained from 2D-FE and 3D-FE analyses were within 10% of one another for footings reinstalled at $\beta>1.0$ from footprints TA and TB in non-homogeneous soil. However, for reinstallation near the steepest and deepest footprint TC, significant over-estimation occurred for the 2D-FE analyses.

E.4 REFERENCES

APPENDIX F. DETAILED DESIGN CALCULATION, FABRICATION AND CALIBRATION OF THE ANGULAR ACTUATOR

F.1 MAXIMUM DESIGN LOAD

The angular actuator was designed to resist the vertical, horizontal and moment loads generated from the jack-up leg penetrating/extracting from the soil sample. Cassidy et al. (2009) presented a maximum horizontal force at load reference point, $H_{LRP}$ of 2MN and maximum moment at load reference point $M_{LRP}$ of 40MN (prototype scale) which are the highest compared to Gaudin et al. (2007) and the results in Chapter 3. Therefore, the results were adopted as the design working load. The angular actuator was designed to operate in 1:100 scale, the loading were scaled down to $H_{LRP}$ and $M_{LRP}$ of 200N and 40Nm respectively. When the pivot point is assumed to be fully-fixed, maximum load will be resulted in the actuator. The maximum load acting at the critical components were determined as shown in Table F.1.
Table F.1: Maximum Load at Critical Elements

<table>
<thead>
<tr>
<th>Loading @ Pivot Point</th>
<th>Loading @ Motor Output Shaft</th>
<th>Loading @ Cable</th>
<th>Loading @ Pivot Point</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_{cable} = 141,\text{N}$</td>
<td>$T_{cable} = 141,\text{N}$</td>
<td>$T_{cable} = 141,\text{N}$</td>
<td>$T_{cable} = 141,\text{N}$</td>
</tr>
<tr>
<td>$H_{LRP} = 200,\text{N}$</td>
<td>$H_{LRP} = 200,\text{N}$</td>
<td>$H_{LRP} = 200,\text{N}$</td>
<td>$H_{LRP} = 200,\text{N}$</td>
</tr>
<tr>
<td>$MLRP = 40,\text{Nm}$</td>
<td>$MLRP = 40,\text{Nm}$</td>
<td>$MLRP = 40,\text{Nm}$</td>
<td>$MLRP = 40,\text{Nm}$</td>
</tr>
<tr>
<td>120.95,\text{mm}</td>
<td>285,\text{mm}</td>
<td>285,\text{mm}</td>
<td>120.95,\text{mm}</td>
</tr>
</tbody>
</table>

Appendix F: Detailed Design Calculation, Fabrication and Calibration of the Angular Actuator
**F.2 MAXIMUM SPEED AND PRECISION REQUIREMENT**

The angular actuator should also move fast enough to accurately model the failure mechanism. The speed requirement was determined from previous centrifuge modelling tests results on jack-up reinstallation. As shown in Chapter 3, in Gaudin et al. (2007) and Cassidy et al. (2009), the horizontal force and moment increased gradually from touchdown level and reached peak values at or near the base of the footprint cavity (between z/D=0.1 to 0.33). Assuming the angular movement is largest at this peak loading, the actuator was therefore designed to rotate the maximum angle (2°) within 0.1D of penetration (6mm for a 60mm diameter footing). For a vertical penetration rate of 0.1mm/s, it would take 60s to reach 6mm penetration depth. The angular speed requirement is therefore 0.03°/s.

Since a relatively small angular movement is anticipated (2°), the actuator must be able to drive the angular movement precisely. The precision can be indicated by the “movement per each motor count” parameter. The angular actuator was designed to have a precision of similar order with the existing linear actuator (2.84E-5mm/count). Figure F.1 shows the design calculation and the precision requirement is in the order of 1.4E-5°/count at the pivot point.

![Figure F.1: Determination of Angular Precision Criteria at Pivot Point](image-url)


F.3 DETAILED DESCRIPTIONS ON CRITICAL COMPONENTS

A DC servo motor module comprised of a Maxon RE-40-150W Graphite Brushes Motor, a 546:1 Maxon GP42C Planetary GearHead gearbox and a 2000 count Maxon HEDS 5540 500CPT 3 channels encoder was selected to fulfil the speed, torque and precision requirements. The selected motor module with precision of 1.49E-5°/count which is in the same order with the requirement was considered to be sufficient. The precision requirement was the critical motor selection criteria.

The cable was designed to withstand the design working load with a minimum factor of safety of 1.5, hence 225N. The 1.0mm diameter strained stainless steel wire with a maximum tensile capacity of 1700N was selected. This was significantly higher than the design requirement. This is because the cable was selected for its flexibility to form a tight loop around the pulleys, rather than based on strength. The cable was pre-tensioned to 300N before use, so as to reduce the potential slack in the pulley system. This reduced the tensile capacity to 1400N. With such a tight pulley system, the rotational movement at the motor output shaft could be accurately transferred to angular movement at the pivot point. In order to monitor and to adjust the pre-tension level in the cable, the cable was connected to a strain gauge at each end.

The cable was firmly connected to the arm through a nut-screw connection at the end of the cable. The arm was designed to rotate around the pivot point. The pivot point was comprised of a series of miniature ball bearing and needle bearing to provide smooth rotational motion under the design load. To prevent out-of-range movement, two limit switches were installed on the inner wall of the actuator box. If tripped the motor immediately stops ensuring no damage is pulley systems.

The output shaft of the motor module was attached to an actuator box which contains all the other components. The actuator box was designed with a transition piece allowing the actuator to be rigidly bolted to the tower of the existing linear actuator unit.

F.4 FABRICATION

The actuator was fabricated and assembled at the Civil Engineering Workshop of UWA. The pulleys were fabricated in nickel coated brass because it is more durable to abrasive action from the loading and unloading of the stainless steel cable loop. The other rotating components (motor output shaft and pivot point) are stainless steel which is durable to the rotary motion. The remainder of the components were fabricated in aluminium so as to minimise the weight. The fabrication process is presented in Figure F.2 and Figure F.3.
Appendix F: Detailed Design Calculation, Fabrication and Calibration of the Angular Actuator

Figure F.2: Various Components of the Angular Actuator
Appendix F: Detailed Design Calculation, Fabrication and Calibration of the Angular Actuator

Figure F.3: Fabrication of Angular Actuator
F.5 SOURCE OF ERROR AND CALIBRATION

Like any motor system, the selected motor module is subjected to back-lash when the motor reverses the direction of driving. Based on the manufacturer’s specification, the theoretical back-lash at the pivot point is 0.026°. This is considered to be insignificant compared to the design angular stroke.

The stainless steel cable also extends when subjected to tension load. The theoretical extension of a 290mm long cable under working load (141N) is 0.1mm. This corresponds to an angular movement of 0.06° at the pivot point. Again, this error is around 3% of the maximum angular movement and was considered to be acceptable.

In order to accurately assess the actual angular movement at the pivot point, an additional encoder (named the front encoder) was mounted on top of the pivot point. This front encoder is free from the back-lash and cable extension error. The front encoder (80000 count Avago AEDA-3300-BE1 encoder) has a precision of 4.50E-3°/count which was the highest available in the market at the time of design. However, the precision is still substantially less than the design precision requirement (1.4E-5°/count). Therefore, the angular movement was still prescribed through the motor module with higher precision and the front encoder was only used for validation of the magnitude of error and for monitoring purposes.

The new actuator was calibrated for accuracy in prescribed angular movement. The vertical position, that of zero rotation ($\theta=0$), was first determined. Slip gauges which are stainless steel blocks with precisely known thickness were used. The actuator was driven until the gap between the arm and the side of the box was exactly the same at the two end of the arm (Figure F.4). At the position the arm is in the exact vertical position (horizontal in the figure) and zero rotation at the pivot point can be assumed.

Following this the accuracy of the rotation of the leg was assessed by physical measurement and by direct comparison with the front encoder reading. As shown in
Appendix F: Detailed Design Calculation, Fabrication and Calibration of the Angular Actuator

Figure F.5, slip gauges, parallel bars (pairs of stainless steel plates with identical thickness) and sine bar (precisely known height and length) were used in the physical assessment. The maximum anticipated angular movement of $\approx 2^\circ$ was considered. The angular actuator was firstly driven from the absolute zero position to 1.9988°, the sine bar, parallel bar and slip gauges was used to determine the height at the two ends of the arm. The physical angular reading was then determined using basic trigonometry. As shown in Table F.2, the difference between the angular reading from the motor encoder and the physically measured angle was less than 0.01°, or 0.6% of that recorded.

<table>
<thead>
<tr>
<th>Angular Reading from Motor Encoder (°)</th>
<th>Length of Sine Bar (mm)</th>
<th>Height of Slip Gauges (mm)</th>
<th>Measured Angle (°)</th>
<th>Difference (°)</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.9988</td>
<td>100</td>
<td>3.47</td>
<td>1.9886</td>
<td>0.0102</td>
<td>0.510</td>
</tr>
</tbody>
</table>

The accuracy of the angular movement was subsequently assessed by evaluating the different reading from motor driving and the front encoder. The front encoder provides accurate angular reading as it is free from the back-lash and cable extension error.
To access the variation of accuracy over the full range of angular movement, the actuator was firstly driven from 0° to the maximum positive stroke of 3°. The actuator was then reversed and moved back to the zero position. Both the motor driving and the front encoder reading were reset to zero to eliminate the effect of back-lash. Finally the actuator was driven from 0° to the maximum negative stroke of -3°. As shown in Figure F.6, the difference between the motor driving and the front encoder readings increases with angle. The maximum different is within 0.7% (Table F.3). The reading from the two encoders is therefore in excellent agreement.

![Comparison of Angular Reading from Motor and from Front Encoder](image)

**Figure F.6: Comparison of Angular Reading from Motor and from Front Encoder**

<table>
<thead>
<tr>
<th>Angular Reading from Motor Driving (°)</th>
<th>Front Encoder Reading (°)</th>
<th>Difference (°)</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>+3.000</td>
<td>+3.020</td>
<td>0.020</td>
<td>0.67</td>
</tr>
<tr>
<td>-3.000</td>
<td>-2.995</td>
<td>0.005</td>
<td>0.17</td>
</tr>
</tbody>
</table>

The effect of backlash was assessed by driving and reversing the actuator in 0.25° increment, over the full range of angular stroke (+/-3°). Figure F.7 shows an example of comparison between motor driving and front encoder readings. The maximum difference during the reverse of angular movement direction is 0.025°, which is in the similar to the manufacture’s specification (0.026°).
Figure F.7: Example Comparison of Angular Reading When Actuator Reversed Driving Direction

F.6 REFERENCES
