DEVELOPMENT OF DESIGN APPROACHES FOR DYNAMICALLY INSTALLED ANCHORS VALIDATED THROUGH FIELD AND CENTRIFUGE STUDIES

by

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DECLARATION FOR THESIS CONTAINING PUBLISHED WORK
AND/OR WORK PREPARED FOR PUBLICATION

This thesis is submitted as a series of journal papers following the regulations of the University of Western Australia. The thesis comprises published work and work submitted for publication which has been co-authored. The bibliographical details of the work and where it appears in the thesis are outlined below. The contribution of the candidate for each of the papers is also given.


The candidate designed the centrifuge tests and carried out all physical modelling experiments and interpreted all data. The candidate compiled a detailed first draft that was revised after comments from both co-authors.

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The stated candidate contributions have been approved by the co-authors of each paper. The co-authors grant permission to include the papers in this thesis.

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Abstract

The performance of dynamically installed anchors is strongly dependent on their embedment depth in the seabed and the angle of load inclination made by the mooring line. Embedment depths for these anchors are notoriously difficult to predict, due to the very high penetration velocities (up to ~30 m/s) that result in complex resistance forces. Prior to this work, the calibration and validation of theoretical embedment models were typically limited to reduced scale centrifuge studies and a limited number of field studies. For both cases, data were restricted to known starting and end conditions: the measured anchor impact velocity at the mudline and the final embedment depth achieved. This means that calibration and/or validation of the complex mechanisms that occur during dynamic embedment could not be verified. Anchor holding capacity predictions were generally based upon a framework used for offshore driven piles in vertical tension, despite mooring lines typically arriving at an angle to the anchor’s vertical axis. Furthermore, little was known with respect to the potential for the hole created by the high speed anchor during dynamic soil penetration to remain open with studies having to speculate on hole closure mechanisms in embedment depth and capacity calculations.

The work carried out to compile this thesis included an experimental study involving reduced scale field tests and centrifuge tests on a typical four fluke dynamically installed anchor configuration. The field research, carried out in a soft soil lakebed environment, yielded recoverable dynamic embedment data from over 100 installations over a large range of drop heights (and hence impact velocities) and a further 89 pull-out capacity tests over a range of load inclinations. The centrifuge research yielded further installation data in clay samples of varying overconsolidation ratios.

In both sets of installation tests, the anchor was instrumented with accelerometers and (for the field tests) rate gyroscopes that allowed a continuous time history of the anchor’s motion to be established over the full dynamic event (free-fall through water and soil embedment). The motion data are used to rigorously calibrate and validate theoretical embedment models, which was possible in this study due to the motion data
that was measured during dynamic installation. Back calculated strain rate factors from
the centrifuge data indicate that current strain rate relationships tend to become
inaccurate over several orders of magnitude increase from the reference strain rate with
the mobilised soil shear strength increasing more rapidly than can be fitted using a
common power law. This leads to the formulation of a new equation for increasing the
strain rate factor so that the power law may be applied to the very high strain rates
observed in dynamic centrifuge tests. The field data considered in this thesis allowed for
a new Release-to-Rest model for dynamically installed anchors to be formulated and
validated. This model considers the motion of the anchor from the point of release in the
water column, modelling the drag resistance that acts on the anchor and its mooring
line. This is necessary as the drag developed on a mooring line is shown to significantly
reduce the anchor velocity as it arrives at the seabed. Embedment predictions for both
the field and centrifuge tests reveal that strain rate dependency may be higher for
frictional resistance than for bearing resistance, although only if a soil strength lower
than the fully remoulded strength is considered as the reference strength, suggesting that
water may be entrained along a boundary layer at the anchor-soil interface.

Centrifuge tests also included dynamic installations of flukeless projectiles with varying
aspect ratios. High speed video observations show, for the first time, that hole closure
may occur at the same rate as the high speed projectile penetrates, or may remain open,
either fully or partially. Any hole closure occurrence is found to be caused by soil
backflow at the rear of the projectile, regardless of aspect ratio. Analyses shows that
hole closure is controlled by a dimensionless strength ratio and an expression is given to
verify hole closure assumptions made in embedment/capacity analyses.

Ultimate vertical capacity data recorded in the field tests help to validate the suitability
of a conventional approach for offshore driven piles over the range of final embedment
depths achieved through dynamic installation. The interaction between vertical and
horizontal (and hence moment) capacities is examined using the inclined loading field
data coupled with large deformation finite element analyses. This forms the basis for a
new and simple design procedure for scaling a dynamically installed anchor subjected to
inclined loading.
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Colm O'Beirne
July 2016
Statement of Candidate Contribution

This thesis is composed of my original work, and contains no material previously published or written by another person except where due reference has been made in the text.

I have obtained the permission of all necessary authors to include the papers in this thesis.

Signature:

Colm O’Beirne

July 2016
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A            contact area
a, b, c, d, e, f fitting constants
A_{bx}      acceleration measurement coincident with the body frame x-axis
A_{by}      acceleration measurement coincident with the body frame y-axis
A_{bz}      acceleration measurement coincident with the body frame z-axis
a_{bz}      linear acceleration coincident with the body frame z-axis
A_{l}       mooring line contact area
a_{linear}  linear acceleration
A_{p}       anchor projected area
A_{s}       anchor frictional area
A_{x}       acceleration measurement coincident with the inertial frame x-axis
a_{x}       linear acceleration coincident with the inertial frame x-axis
A_{y}       acceleration measurement coincident with the inertial frame y-axis
a_{y}       linear acceleration coincident with the inertial frame y-axis
A_{z}       acceleration measurement coincident with the inertial frame z-axis
a_{z}       linear acceleration coincident with the inertial frame z-axis
C_{D}       drag coefficient
C_{D,a}     anchor drag coefficient
C_{D,l}     trailing line drag coefficient
c_{h}       coefficient of horizontal consolidation
D           diameter of anchor, projectile, penetrometer or hole
d           T-bar diameter
D_{ball}    diameter of ball penetrometer
<table>
<thead>
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<th>Symbol</th>
<th>Definition</th>
</tr>
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<tbody>
<tr>
<td>$d_{eff}$</td>
<td>effective anchor diameter</td>
</tr>
<tr>
<td>$E_{total}$</td>
<td>total energy</td>
</tr>
<tr>
<td>$e_z$</td>
<td>distance between the padeye and anchor centroid</td>
</tr>
<tr>
<td>$F_b$</td>
<td>buoyancy force</td>
</tr>
<tr>
<td>$F_{bear}$</td>
<td>bearing resistance</td>
</tr>
<tr>
<td>$F_d$</td>
<td>drag resistance</td>
</tr>
<tr>
<td>$F_{frict}$</td>
<td>frictional resistance</td>
</tr>
<tr>
<td>$F_g$</td>
<td>geotechnical resistance</td>
</tr>
<tr>
<td>$F_i$</td>
<td>inclined capacity</td>
</tr>
<tr>
<td>$F_{net}$</td>
<td>net force</td>
</tr>
<tr>
<td>$F_v$</td>
<td>vertical capacity</td>
</tr>
<tr>
<td>$g$</td>
<td>acceleration due to gravity</td>
</tr>
<tr>
<td>$H_{max}$</td>
<td>maximum horizontal capacity</td>
</tr>
<tr>
<td>$k$</td>
<td>undrained shear strength gradient with depth</td>
</tr>
<tr>
<td>$L$</td>
<td>anchor, projectile length</td>
</tr>
<tr>
<td>$L_{fluke}$</td>
<td>fluke length</td>
</tr>
<tr>
<td>$L_{shaft}$</td>
<td>shaft length</td>
</tr>
<tr>
<td>$L_{tip}$</td>
<td>tip length</td>
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<tr>
<td>$m'$</td>
<td>added mass</td>
</tr>
<tr>
<td>$m$</td>
<td>mass</td>
</tr>
<tr>
<td>$M_{max}$</td>
<td>maximum rotational capacity</td>
</tr>
<tr>
<td>$M_{ult}$</td>
<td>ultimate rotational capacity</td>
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<tr>
<td>$N$</td>
<td>acceleration level</td>
</tr>
<tr>
<td>$n, n_1$</td>
<td>strain rate parameters</td>
</tr>
<tr>
<td>$N_{ball}$</td>
<td>bearing capacity factor for spherical penetrometer</td>
</tr>
<tr>
<td>$N_c$</td>
<td>bearing capacity factor</td>
</tr>
<tr>
<td>$N_{kt}$</td>
<td>piezocone bearing factor</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>$q_{\text{net}}$</td>
<td>net penetration resistance</td>
</tr>
<tr>
<td>$r$</td>
<td>radius</td>
</tr>
<tr>
<td>$R_f$</td>
<td>strain rate function</td>
</tr>
<tr>
<td>$R_{f,\text{bear}}, R_{f,\text{frict}}$</td>
<td>strain rate functions for bearing and frictional resistance</td>
</tr>
<tr>
<td>$R_x$</td>
<td>roll matrix</td>
</tr>
<tr>
<td>$R_y$</td>
<td>pitch matrix</td>
</tr>
<tr>
<td>$R_z$</td>
<td>yaw matrix</td>
</tr>
<tr>
<td>$R'_b$</td>
<td>direction cosine matrix (body frame to inertial frame)</td>
</tr>
<tr>
<td>$s$</td>
<td>distance travelled in the direction of motion</td>
</tr>
<tr>
<td>$s_0$</td>
<td>initial distance travelled in the direction of motion</td>
</tr>
<tr>
<td>$s_{bz}$</td>
<td>vertical distance travelled coincident with the body-frame</td>
</tr>
<tr>
<td>$S_t$</td>
<td>soil sensitivity</td>
</tr>
<tr>
<td>$s_u$</td>
<td>undrained shear strength</td>
</tr>
<tr>
<td>$s_{um}$</td>
<td>undrained shear strength at mudline</td>
</tr>
<tr>
<td>$s_z$</td>
<td>vertical distance travelled coincident with the inertial-frame</td>
</tr>
<tr>
<td>$s_{z0}$</td>
<td>initial vertical distance travelled coincident with the inertial-frame</td>
</tr>
<tr>
<td>$T$</td>
<td>non-dimensional time</td>
</tr>
<tr>
<td>$T'_b$</td>
<td>angular velocity transformation matrix (body frame to inertial frame)</td>
</tr>
<tr>
<td>$t$</td>
<td>time</td>
</tr>
<tr>
<td>$t_{\text{fluke}}$</td>
<td>fluke thickness</td>
</tr>
<tr>
<td>$v$</td>
<td>velocity</td>
</tr>
<tr>
<td>$v_{bz}$</td>
<td>velocity coincident with the body frame z-axis</td>
</tr>
<tr>
<td>$v_i$</td>
<td>impact velocity</td>
</tr>
<tr>
<td>$V_{\text{ult}}$</td>
<td>ultimate vertical capacity</td>
</tr>
<tr>
<td>$v_x$</td>
<td>velocity coincident with the inertial frame x-axis</td>
</tr>
<tr>
<td>$v_{x0}$</td>
<td>initial velocity coincident with the inertial frame x-axis</td>
</tr>
<tr>
<td>$v_y$</td>
<td>velocity coincident with the inertial frame y-axis</td>
</tr>
</tbody>
</table>
### List of Symbols

- \( v_{y0} \): initial velocity coincident with the inertial frame y-axis
- \( v_z \): velocity coincident with the inertial frame z-axis
- \( v_{z0} \): initial velocity coincident with the inertial frame z-axis
- \( W \): anchor dry weight
- \( w_{fluke} \): fluke width
- \( W_s \): submerged anchor weight
- \( x \): inertial frame x-axis
- \( x_b \): body frame x-axis
- \( y \): inertial frame y-axis
- \( y_b \): body frame y-axis
- \( z \): depth, anchor tip position relative to the mudline, inertial frame z-axis
- \( z_b \): body frame z-axis
- \( z_{deep} \): transitional depth
- \( z_e \): final projectile embedment depth (from mudline to rear of projectile)
- \( z_{e,tip} \): anchor tip embedment depth
- \( \alpha \): interface friction ratio
- \( \beta, \beta_{min}, \beta_{max} \): strain rate parameters
- \( \gamma' \): effective unit weight
- \( \gamma \): unit weight of soil
- \( \Delta \theta \): difference between body frame and inertial frame pitch angles
- \( \Delta \varphi \): difference between body frame and inertial frame roll angles
- \( \theta \): angle of load inclination to the horizontal, pitch angle coincident with the inertial-frame
- \( \theta_{acc} \): pitch angle coincident with the inertial-frame (derived from accelerometer measurements)
- \( \theta_b \): pitch angle coincident with the body-frame
- \( \theta_{b0} \): initial pitch angle coincident with the body-frame
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\theta_m$</td>
<td>mooring line angle at the mudline</td>
</tr>
<tr>
<td>$\Lambda$</td>
<td>plastic volumetric strain ratio</td>
</tr>
<tr>
<td>$\rho$</td>
<td>density</td>
</tr>
<tr>
<td>$\rho_s$</td>
<td>soil density</td>
</tr>
<tr>
<td>$\sigma'v$</td>
<td>vertical effective stress</td>
</tr>
<tr>
<td>$\phi$</td>
<td>roll angle coincident with the inertial-frame</td>
</tr>
<tr>
<td>$\phi_{acc}$</td>
<td>roll angle coincident with the inertial-frame (derived from accelerometer measurements)</td>
</tr>
<tr>
<td>$\phi_b$</td>
<td>roll angle coincident with the body-frame</td>
</tr>
<tr>
<td>$\phi_{b0}$</td>
<td>initial roll angle coincident with the body-frame</td>
</tr>
<tr>
<td>$\psi$</td>
<td>yaw angle coincident with the inertial-frame</td>
</tr>
<tr>
<td>$\psi_b$</td>
<td>yaw angle coincident with the body-frame</td>
</tr>
<tr>
<td>$\psi_{b0}$</td>
<td>initial yaw angle coincident with the body-frame</td>
</tr>
<tr>
<td>$\omega$</td>
<td>angular velocity</td>
</tr>
<tr>
<td>$\omega_{bx}$</td>
<td>rotation rate about the body-frame $x$-axis</td>
</tr>
<tr>
<td>$\omega_{by}$</td>
<td>rotation rate about the body-frame $y$-axis</td>
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<tr>
<td>$\omega_{bz}$</td>
<td>rotation rate about the body-frame $z$-axis</td>
</tr>
<tr>
<td>$\omega_x$</td>
<td>rotation rate about the inertial-frame $x$-axis</td>
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<tr>
<td>$\omega_y$</td>
<td>rotation rate about the inertial-frame $y$-axis</td>
</tr>
<tr>
<td>$\omega_z$</td>
<td>rotation rate about the inertial-frame $z$-axis</td>
</tr>
<tr>
<td>$\dot{\gamma}_{ref}$</td>
<td>reference strain rate</td>
</tr>
<tr>
<td>$\dot{\gamma}$</td>
<td>strain rate</td>
</tr>
<tr>
<td>$\Delta$</td>
<td>mooring line displacement</td>
</tr>
<tr>
<td>$\Delta z_{scrape}$</td>
<td>sample scrape depth</td>
</tr>
<tr>
<td>$\mu$</td>
<td>friction coefficient, resultant tilt angle (relative to gravity)</td>
</tr>
</tbody>
</table>
Abbreviations

AVTM angular velocity transformation matrix
DCM direction cosine matrix
DEPLA dynamically embedded plate anchor
DoF degrees of freedom
DPA deep penetrating anchor
FPSO Floating, Production, Storage and Offloading
GoM Gulf of Mexico
IFFS instrumented free-falling sphere
IMU inertial measurement unit
LDFE Large Deformation Finite Element
MEMS Micro Electro Mechanical System
MODU Mobile Offshore Drilling Unit
OCR Over Consolidation Ratio
PERP Photo Emitter Receiver Pair
R2R release to rest
ROV remotely operated vehicle
SEPLA suction embedded plate anchor
VLA vertically loaded anchor
Chapter 1  Introduction

1.1  Offshore energy; shallow to deep

As shallow water reserves become exhausted and global energy demands continue to rise (see Figure 1.1), the offshore petroleum industry has set out to conquer deep-waters. This extreme environment has emerged as a key source for new oil and gas reserves in recent times. The definition of deep-water has evolved with technology, but nowadays water depths less than 500 m are typically considered shallow, with depths of 500 – 1500 m representing deep-water and depths greater than 1500 m classified as ultra-deep (Colliat 2002; Randolph and Gourvenec 2011).

A good example to exemplify the transition from shallow to deep-water by the offshore energy industry is to consider oil and gas production in the Gulf of Mexico over the last two decades (see Figure 1.2). In 1990, approximately four percent of the oil and less than one percent of the natural gas produced here came from deep-water areas. By 2003, more than 60 percent of the oil and 29 percent of the natural gas was produced from deep and ultra-deep waters (Cleveland, 2010). By the end of 2009, less than 20 percent of the total annual oil production was recovered through shallow water activities; deep-water and ultra-deep accounting for 48 and 32 percent respectively (EIA, 2010).

Venturing into these deeper waters is being made achievable through advances in drilling/production platforms, foundation technology and storage facilities. The increased operating depths are typically coupled with steep rises in installation and procurement costs, making traditional platforms which are directly fixed to the seabed prohibitively expensive. Floating facilities, including Semisubmersibles/Tension Leg platforms/Spar platforms/Floating production, storage and offloading systems (see Figure 1.3), held by reliable mooring systems are thus required.
Figure 1.1 Statistical review and forecast of world energy consumption (after BP, 2015)

Figure 1.2 Gulf of Mexico federal oil production correlated to water depths (EIA, 2010)
1.2 Dynamically installed anchors

The cost and complexity of installing conventional offshore anchors in deep and ultra-deep water has led to the design of new and innovative anchoring solutions. Dynamically installed anchors are one such solution, developed to regulate the procurement and installation costs of mooring systems at a reasonable level, without decreasing reliability. They are rocket or torpedo shaped, typically consisting of a cylindrical shaft 12 – 15 m long and 0.8 – 1.2 m in diameter, and four flukes at the trailing edge. They are released from a specified height above the seabed (typically 50 – 100 m) and penetrate to a target depth below the mudline through the kinetic energy obtained through free-fall (see Figure 1.4). Installation costs are low relative to other deep/ultra-deep water anchors due mainly to simplicity; the free-fall process being
relatively independent of water depth and requiring no external energy source or mechanical intervention. Medeiros (2001) reported related cost savings of 30% when also accounting for the lighter weight and inexpensive fabrication compared with conventional anchoring systems. The anchors can withstand both horizontal and vertical load components and are thus suitable for both catenary and taut-leg mooring systems. Resistance to loading is provided through bearing and frictional resistance developed along the anchor-soil interface.

Dynamically installed anchors have been advocated as offering the most cost effective and uncomplicated mooring solution to the offshore energy (e.g. Ehlers et al. 2004). However, uncertainty regarding the prediction of final embedment depth and subsequent holding capacity remains. The former is complicated by the very high strain rates evoked at the soil-anchor interface resulting from the high penetration velocities and the poorly understood soil response in the wake of the advancing anchor. A more thorough analysis of dynamically installed anchors is given at the beginning of 0.

![Figure 1.4 Dynamically installed anchor installation concept (after Lieng et al. 1999)](image-url)
1.3 Research objectives

The main objective of this research was to develop calibrated and validated design tools for predicting the performance of dynamically installed anchors in soft soil. Encompassed within this main objective are four specific sub-objectives:

1. Develop an experimental database, encompassing both centrifuge and field tests, for dynamically installed anchors as a means of establishing anchor performance in soft soil and for calibrating design tools.

2. Answer the question as to how the soil responds in the wake of a dynamically installed anchor during dynamic embedment, establishing the conditions under which the hole formed by the passage of the anchor may close.

3. Refine and calibrate analytical embedment models for dynamically installed anchors using motion data collected in the centrifuge and field tests from sub-objective one.

4. Establish design tools for predicting the monotonic capacity of dynamically installed anchors for a range of load inclinations, calibrated using field data from sub-objective one.

1.4 Chapter organisation and thesis map

This thesis comprises eight chapters and is presented as a series of academic journal papers that have either been published or submitted for publication. An outline of each chapter, including a short description of how the chapter helps to form the thesis in its entirety, is given below.

Chapter 2 reviews the important literature of dynamically installed anchors and includes necessary background information for developing the thesis. Shortcomings in the existing knowledge are highlighted through the review, leading to the establishment of the four specific sub-objectives previously given in section 1.3 that are addressed in chapters 3 – 7. Details of varying dynamically installed anchor concepts and their respective use to date are initially given. An analytical embedment model, which has been adopted over a range of dynamic studies, is then discussed. The complex forces that the model must compute are examined; particularly fluid mechanics drag resistance and strain-rate-dependent shearing resistance. This is followed by an overview of the
poorly understood soil response during high speed installations, specifically the hole created by an advancing projectile. Finally, a review of important considerations for estimating the capacity of dynamically installed anchors is given which includes insight into consolidation effects.

As the thesis is prepared as a series of journal papers, additional literature review specific to each publication is included towards the beginning of each individual chapter.

Chapter 3 addresses sub-objective 2 through observations and analysis from a centrifuge testing program of model dynamically installed anchors and cylindrical projectiles of varying aspect ratio penetrating soft soil (using data from sub-objective 1). A high-speed video camera captures the impact and early penetration event in soil samples with varying mudline strengths and overconsolidation ratios. The testing program provides significant new understanding of the soft soil response in the wake of dynamically installed projectiles. This understanding is subsequently incorporated into theoretical embedment prediction models presented in Chapters 4 and 6. The procedures and outcomes from the testing program have been published:


Chapter 4 addresses sub-objective 3 through a further centrifuge testing program in soil samples with varying mudline strengths and overconsolidation ratios (using data from sub-objective 1). A model dynamically installed anchor is fitted with an internal micro-electro mechanical system (MEMS) accelerometer that tracks the anchor’s motion during free-fall and embedment. The MEMS data was used to calibrate and ultimately reinforce the merit of an analytical embedment model while recognising the soil response findings presented in Chapter 3. Appropriate strain rate parameters to be used for dynamic tests in high acceleration testing environments (i.e. the centrifuge) were also calibrated. Coupled with the findings from Chapter 3, this chapter aids the development of Chapter 6 via complete experimental motion data profiles obtained in a controlled environment. The procedures and outcomes from the testing program have been accepted for publication:
Chapter 1


Chapter 5 helps to address sub-objective 3 (using data from sub-objective 1) by describing the technical development of an inertial measurement unit (IMU), crucial for the correct analysis of field testing presented in Chapter 6. A comprehensive framework for interpreting the motion data recorded by the device is described and validated against direct measurements taken from a range of field tests on two different types of dynamically installed anchor and a free-fall sphere penetrometer. The framework is adopted in Chapter 6. The procedures and outcomes from the motion data analyses have been published:


Chapter 6 addresses sub-objective 3 (using data from sub-objective 1) and expands on the reduced scale model testing presented in Chapter 4 through a series of field tests. Data captured from dynamic installation tests with a 1:20 scale anchor (instrumented with the IMU described in Chapter 5) in a lake in Northern Ireland are analysed. Together with the analysis of an additional series of field tests reported by industry using a 1:3 scale anchor in the North Sea, important insight is made into the hydrodynamic forces acting on both the anchor and trailing line. This insight could not be gained from the centrifuge tests described in Chapters 3 and 4. A new Release-to-Rest model which simulates the motion history of the anchor during freefall in water and dynamic embedment in soil is given and verified against the entire field database. The procedures and outcomes from the chapter have been accepted for publication:

Chapter 7 addresses objective 4 through an investigation on the capacity characteristics of the 1:20 scale model anchor described in Chapter 6 (using field capacity data from sub-objective 1). Monotonic loading of the embedded anchor in the same lake in Northern Ireland gives insight into the holding capabilities of dynamically installed anchors. Experimental vertical capacities are compared with conventional calculations for offshore driven piles in soft soil. Capacities are also established over a range of load inclinations for a constant embedment depth in the field. Corresponding large deformation finite element analysis is used in unison with the field observations to establish a yield envelope of vertical and moment components. This forms the basis of a simple design procedure for estimating the required size of a dynamically installed anchor for a given mooring design. The procedures and outcomes from the field tests and numerical analyses examining the capacity characteristics have been published:


Chapter 8 summarises the major research findings from Chapters 3 to 7.

The thesis map on Figure 1.5 has been compiled to help navigate the thesis contents. Arrows on the thesis map indicate which sub-objectives and preceding chapters correspond or help to compile each chapter of the thesis. Highlighted versions of the thesis map are given at the beginning of chapters 3 to 7, indicating the chapter context within the thesis.
Main objective: Develop calibrated and validated design tools for predicting the performance of dynamically installed anchors in soft soil

Chapter 1: Brief introduction of the problem and a description of the objectives and structure of the thesis

Chapter 2: Literature review providing a basis for assessing the performance of dynamically installed anchors and outlining the current shortcomings to be addressed in chapters 3 to 7

Chapter 3: Soil response in the wake of dynamically installed projectiles

Chapter 4: Assessing the penetration resistance acting on a dynamically installed anchor in normally and over consolidated clay

Chapter 5: In situ measurement of the dynamic penetration of free-fall projectiles in soft soils using a low-cost inertial measurement unit

Chapter 6: A release-to-rest model for dynamically installed anchors

Chapter 7: Capacity of dynamically installed anchors as assessed through field testing and three dimensional large deformation finite element analyses

Chapter 8: Conclusions derived from the work described in the thesis

Completion of the above sub-objectives leads to the completion of the main objective and Chapter 8

Figure 1.5 Thesis map
Chapter 2  Literature Review

2.1 Introduction

This chapter reviews the relevant literature of dynamically installed anchors and includes necessary background information for addressing the research objectives as set out in Chapter 1. The chapter is categorised into four main sections. The first section explores varying dynamically installed anchor concepts and their available performance to date. The second section outlines an analytical embedment model that features heavily in Chapters 4 and 6. The complex forces that the model must compute are examined, particularly fluid mechanics drag resistance and strain-rate-dependent shearing resistance. The third section focuses on the poorly understood soil response during high speed installations which is the primary topic of Chapter 3. The fourth section reviews aspects for estimating the capacity of dynamically installed anchors, mainly relevant to Chapter 7.

The chapter aims to demonstrate the merits as well as highlight the shortcomings of the current knowledge and corresponding tools/formulae for predicting the performance of dynamically installed anchors. The shortcomings are then addressed through the thesis.
2.2 Shortcomings of traditional deep-water anchoring systems

Three main types of embedded anchor have traditionally been used for securing offshore floating facilities: driven or drilled and grouted piles; suction caissons; drag anchors (conventional fixed fluke or plate). More recently developed anchors, directly aimed at making ultra-deep sea operations more practical, reliable and cost effective include the suction embedded plate anchor (SEPLA) and dynamically installed anchors. A brief overview of the shortcomings of each anchoring system (excluding dynamically installed anchors) is given below.

Anchor Pile:

Pile driving equipment and large crane barges required for installation result in very high costs, increasing considerably with water depth. The majority of pile driving hammers have a practical operational water depth limit of approximately 1500 m and the complexity of installation at greater depths diminishes the anchor’s popularity for ultra-deep application.

Suction caisson:

The suction caisson is considered a mature anchoring system with well-developed analysis procedures. Anderson et al. (2005) give an exhaustive list of moorings utilising suction caissons, including about 500 installed at more than 50 locations in water depths reaching 2000 m by the end of 2003. It is possible that the thin-walled caisson will buckle due to excessive under-pressure. Considering its large diameter and relatively low penetration depth, scour of the seabed can also compromise the anchor’s capacity. In addition, caissons require a considerable amount of deck space during transport and the use of a heavy lift vessel during installation, resulting in high installation costs.

Drag anchor (fixed fluke):

Fixed fluke anchors are not designed to take large vertical loads and in fact are withdrawn by applying vertical load to the connecting mooring line. They are, therefore, only applicable for catenary mooring configurations and found to be unsuitable for deep-water applications using taut or semi-taut line moorings.
Uncertainty exists over the trajectory and final embedment depth of the anchor during installation even in uniform seafloor soil conditions (Aubeny, et al., 2001) and significant drag distances are often necessary to attain sufficient final embedment depths. Greater site investigation costs and increased likelihood of interference with existing mooring lines and subsea pipelines become unwanted resultant.

Drag anchor (drag-in plate):

Drag in-plate anchors, also referred to as vertically loaded anchors (VLAs), were developed to overcome some of the limitations of the traditional fixed fluke anchor and can be used for taut-leg mooring systems. Upon reaching a predetermined cable tension during installation, the shank or wire bridle is triggered (by means of a shear pin) to allow for rotation/keying so that the fluke becomes normal or almost normal to the anchor line force. VLAs yield similar shortcomings to traditional fixed fluke drag anchors regarding significant drag distances for installation and uncertainty of final embedment depth. There is an additional degree of uncertainty regarding the triggering process and the embedment depth loss due to the keying process.

Suction embedded plate anchor (SEPLA):

SEPLAs are now commonly used for securing mobile offshore drilling units in water depths of up to approximately 2200 m (Brown et al. 2010). Considerable numerical and experimental model studies have helped to make the anchors a well-established concept in the deep-water mooring market with a field proven experience record (Wilde et al. 2001; Gaudin et al. 2006; Song et al. 2005 & 2008).

The anchor installation method aims to overcome the uncertainty of final embedment depth experienced with drag anchor installation. However, the direct embedment plate anchors can still succumb to loss of embedment during the keying process (O’Loughlin et al. 2006; Yang 2010; Song et al. 2009). The installation and retrieval of the follower can also result in a zone of weakened soil extending from the plate to the seafloor surface resulting in potentially lower capacities (Song et al. 2007).
2.3 Dynamically installed anchor concepts; experience and performance data

Although a relatively new concept, numerous dynamically installed anchor designs have already been commercialised. The recent focus is understandable considering the concept appears to offer the most cost effective and uncomplicated mooring solution to the offshore energy industry above all other deep-water anchors. Anchor designs to date exhibit variances in geometry, mass, mooring line attachment features and resistance techniques to loading. Nevertheless, each developer has aimed to exploit the fundamental advantages of the concept which can be grouped into two categories:

1. **Simplicity.** The free-fall installation process results in less complex marine operations compared to other installation practices and hence less marine vessel time. No external source of energy is required for installation and limited use of remotely-operated vehicles is needed. Dynamic installation is also less sensitive to environmental conditions such as water depth. Furthermore, the anchor holding capacity is less sensitive to the initial estimate of the soil shear-strength profile and is rather a function of input energy (O’Loughlin et al. 2004a).

2. **Cost-effectiveness.** The compact size of dynamically installed anchors allows more anchors per trip to be transported compared to other deep-water anchors such as suction caissons. A typical mooring configuration for an offshore floating platform may consist of 12-16 anchors (located in three or four groups) which could be deployable in the course of a single trip with an anchor-handling vessel. Anchor fabrication is also inexpensive with easily sourced material.

The following sections detail the most prominent dynamically installed anchor designs, including their available performance data and experience to date in the field.

**2.3.1 Deep Penetrating Anchor (DPA™)**

Development of the DPA began in the late 1990s (Lieng et al. 1999). The anchor comprises an arrow/dart shape, with a thick-walled steel cylindrical shaft (Figure 2.1). Four clipped delta type flukes (flat plates) with forward swept trailing edges are fixed to the upper shaft section; improving hydrodynamic stability during vertical free-fall and increasing frictional uplift capacity following embedment. A single mooring line trails from a padeye located at the top of the anchor. This padeye position is beneficial for
handling/installation purposes and permits loading in any direction once installed, thereby removing the need for identifying anchor orientation at final embedment.

Subsea intervention is limited to a remotely operated vehicle (ROV) rendering anchor installation almost independent of water depth. The device initiates anchor release into free-fall via transmission of an acoustic signal or simply by cutting a sacrificial sling. The same ROV is subsequently used for observation along the sea floor and monitors the tightening of the permanent mooring line. The anchor is considered appropriate for taut-leg and vertically loaded mooring systems. Initial design requirements included that the concept provide capacities of at least 400-500 tons for short-term loads and 300 tons for long-term static loads (Lieng et al. 2000).

![Proposed DPA configuration](image)

**Figure 2.1 Proposed DPA configuration (Lieng et al. 1999)**

### 2.3.1.1 Field experience

The first reduced scale (1:3) DPA field trials were performed at the Trondheim Fjord, Norway in 2003 (see Figure 2.2a). No data relating to the tests have been published owing to the loss of the only instrumented anchor through failure of the fibre anchor line during extraction (Dragland, 2009). Further 1:3 scale tests were carried out at the Troll Field, Norway in 2008 as reported by Sturm et al. (2011) who offer a limited account.
Chapter 2

The experimental program encompassed 12 dynamic installations in ~ 300 m water depths primarily aimed at examining:

1. Anchor verticality during free-fall through the water column and seabed penetration.
2. Lateral drift relative to the release site.
3. Anchor velocity and the dependence of final embedment depth with drop height.

Two DPAs were used with a dry mass of 2.9 t, length 4.4 m, shaft diameter 0.4 m, fluke width ~ 0.5 m and fluke length 2 m. One DPA featured instrumentation for measuring acceleration, inclination and pore pressure while the other was a ‘dummy’ anchor.

Figure 2.2 (a) Reduced scale (1:3) DPAs tested at the Trondheim Fjord, Norway (Skaugset & Fjeldstad, 2009) and (b) Full scale 80 t DPA installed at the Gjøa Field, Norway
Following drop heights above the seabed of 15 to 75 m, it was observed that 70% of the estimated terminal velocity was achievable from a drop height of approximately 5 times the anchor length (L). Maximum velocity was realised at approximately 8L, and as the anchor free-fell further, its velocity began to reduce.

Impact velocities at the mudline of approximately 13 – 15 m/s gave final tip embedment depths of 1.6 – 2L. Average anchor tilts of 2.8 degrees from the vertical plane were reported, however drop heights greater than 8L led to increases in inclination amplitude. Typical sea current velocities of 0.25 m/s recorded at the drop altitudes were found to have negligible effects on lateral drift during anchor free-fall over the full range of drop heights. Finally, ultimate vertical capacities measured directly after installation (i.e. no consolidation) were in the range 2.3 – 2.6 times the anchor dry weight.

The apparent success of the 1:3 reduced scale tests at the Troll Field led to the installation of two full-sized 80 t DPAs in the Gjøa Field, Norway in August 2009 (Lieng et al. 2010). The 13 m long anchors featuring a shaft diameter of 1.2 m (Figure 2.2b) are to help moor MODUs and are rated with a maximum pull-out capacity of 700 t. Such a high rating, which gives an efficiency of 9 (anchor dry weight divided by maximum pull-out capacity) seems unlikely considering the laboratory performance data given in the next section. Recoverable on-board instrumentation was used in both anchors to gain insight on the dynamic installation process and the findings are given in Table 2.1. Final embedment sites were reportedly within a two meter horizontal radius of the release point for both anchors.

<table>
<thead>
<tr>
<th></th>
<th>DPA 1</th>
<th>DPA 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water depth (m)</td>
<td>~ 360</td>
<td>~ 360</td>
</tr>
<tr>
<td>Drop height (m)</td>
<td>50</td>
<td>75</td>
</tr>
<tr>
<td>Maximum velocity (m/s)</td>
<td>24.5</td>
<td>27</td>
</tr>
<tr>
<td>Sediment characteristics</td>
<td>Stiffer</td>
<td>Softer</td>
</tr>
<tr>
<td>Final tip penetration (m)</td>
<td>24</td>
<td>31</td>
</tr>
<tr>
<td>Maximum anchor tilt</td>
<td>&lt; 2°</td>
<td>&lt; 2°</td>
</tr>
</tbody>
</table>

Table 2.1 Dynamic installation results for two full-scale DPAs deployed in the Gjøa Field after Lieng et al. (2010)
2.3.1.2 Physical modelling performance data

A large suite of centrifuge tests, where prototype stresses and velocities can be replicated, were conducted on 1:200 reduced scale model DPAs in kaolin clay with prototype shear strength gradients in the range \( k = 0.83 - 1.5 \) kPa/m (O’Loughlin et al. 2004a and b; Richardson 2008; Richardson et al. 2009; O’Loughlin et al. 2009). The model anchors featured an ellipsoidal tip, a constant length and diameter of 75 mm and 6 mm respectively across all the studies, however the mass and fluke arrangement were varied. Impact velocities ranged from 0 to ~30 m/s with corresponding final tip embedment depths of approximately 80 to 220 mm (~ 1.1 to 2.9 anchor lengths). The installation results demonstrated that the anchor embedment depth increases with increasing mass and impact velocity (see Figure 2.3a). Interestingly, tests with zero impact velocity achieve relatively high embedment depth, suggesting a strong dependence on anchor mass. The data also show that embedment depth reduces with increasing surface area; shown in Figure 2.3b when comparing different fluke arrangements and also in Figure 2.3a where anchor A4 (m= 5.4 g, no flukes) and anchor A5 (m= 9.6 g, four flukes) show similar embedment trends despite the difference in mass.

Figure 2.3 Dependence of anchor tip embedment depth on: (a) impact velocity, mass and surface area (after O’Loughlin et al. 2013b); (b) impact velocity and surface area (after O’Loughlin et al. 2004a); (c) on aspect ratio (after O’Loughlin et al. 2013b)
Another centrifuge study encompassing a large set of zero fluke anchors featuring hemispherical tips was conducted by Richardson et al. (2006). The anchors varied widely in aspect ratio (L/D = 1 – 14) and mass. Impact velocities of 9 to 20 m/s were achieved leading to prototype tip embedment depths of 1 to 8 times the anchor length. The installation results demonstrated a decrease in embedment depth with increasing aspect ratio for anchors of equivalent mass (Figure 2.3c).

Physical modelling studies have demonstrated an increase in DPA vertical pullout capacity with increasing embedment depth. O’Loughlin et al. 2004b reported anchor efficiencies (defined as the ratio of holding capacity to anchor dry weight) in the range 1.7 to 4.4 following post-installation consolidation periods of approximately 1 prototype year for DPAs featuring varying fluke arrangements (see Figure 2.4).

![Figure 2.4 Increasing DPA vertical capacity with embedment depth; anchor efficiencies in the range 1.7 to 4.4 (after O’Loughlin et al. 2004b)](image)

Richardson et al. 2009 report an extensive suite of vertical capacity tests on model DPAs in the centrifuge following average tip embedment depths of 1.4 L. Anchor efficiencies of approximately 1 to 4 were observed following post-installation consolidation periods of approximately 18 days to 228 years in prototype scale. Interestingly, typical load versus displacement plots for the extractions showed the DPA response to be characterized by a sharp increase in load towards an initial maximum (Peak 1) followed by a sudden drop in load and a subsequent increase towards a secondary maximum capacity (Peak 2) of lower magnitude than Peak 1 (see Figure 2.5a). The response was interpreted to be due to different rates of capacity mobilisation;
Peak 1 corresponding to the high and brittle frictional resistance and Peak 2 corresponding to a more gradual mobilisation of the bearing resistance. Increasing post-installation consolidation times were shown to result in increases to the maximum vertical capacities achievable as shown in Figure 2.5b where the capacities have been normalised by the anchor’s projected area and the average undrained shear strength over the length of the anchor at the final embedment depths. The non-dimensional time factor (used on Figure 2.5b) and the theory of consolidation are discussed further in section 2.6.3.

Figure 2.5 (a) Typical load displacement response of a model DPA during extraction in the centrifuge (after Richardson et al. 2009); (b) Dependence of normalised capacity on post-installation consolidation time (after Richardson et al. 2009)
2.3.2 Torpedo Pile

The Brazilian energy company Petrobras first introduced torpedo piles in 1996 as an inexpensive, easily installed anchor for riser flow-line restraint and patented the concept soon afterward (Medeiros et al. 1997; Medeiros, 2001). Following initial success, with more than ninety installed between January 2000 and December 2001 for flexible risers in the Campos Basin (Medeiros, 2002), the company began to explore torpedo piles’ potential for mooring MODUs and FPSO vessels.

The piles comprise a tubular steel body made of line pipe and feature a conical tip. A single mooring line trails from an omni-directional chain attachment point located at the top of the anchor. Four flukes are typically attached to the body however they are longer and narrower than those featured on the DPA™ (see Figure 2.6). To increase overall mass and enhance verticality during free-fall and soil penetration further, ballasting is used in the tubular anchor body to ensure a low centre of gravity. This is accomplished using sections (ascending from the tip) of lead, cast iron, concrete and scrap chain.

![Figure 2.6 T-98 Torpedo Piles](image)

Although torpedo piles can be designed for specific loads and soil conditions, Petrobras has developed a number of standard torpedo sizes suitable for varying applications which include: T-24s typically used for flow-line restraint; T-43s typically used for MODUs; T-98s typically used for FPSOs. The numbers indicate the dry weight of the anchor in tons. Fluke magnitudes depend on the anchor use with T-24s requiring only very small fins (primarily employed to keep them from rolling on deck of handling vessels). Conversely, very large fins are used on the MODU and FPSO anchors to increase holding capacity once installed.
It has been reported that since 1996, more than 1,000 torpedo piles have been installed for anchoring deep-water flow-lines and facilities offshore Brazil (Acteon, 2009; Wilde, 2009). In the beginning of 2002, the first T-43s were used to moor a deep-water drilling rig and in the subsequent 3 years, more than 50 T-43s had been installed (Brandão et al. 2006). Wilde (2009) refers to Petrobras’ large database of installation information, developed over the years from when the first torpedo piles were instrumented with accelerometers and inclinometers. However, very limited data and analysis has been made available in the public domain.

2.3.2.1 Field experience

Medeiros (2002) provides some insight into full scale field tests performed on torpedo piles in the Campos Basin in water depths of 200 – 1000 m. A 12 m long, 0.763 m diameter flukeless pile, with a dry weight of 400 kN was released into free-fall 30 m above four varying seabeds. Average tip penetrations for the different site locations were given approximately as: 2.4L in normally consolidated clay, 1.1L in overconsolidated clay, 1.25L in uncemented calcareous sand and 1.8L in a seabed of 13 m thick fine sand overlying normally consolidated clay. The depth measurement reported for the sand overlying clay is unexpected considering the much lower embedments measured in the overconsolidated clay and uncemented calcareous sand.

Load capacity tests on two torpedo piles in normally consolidated clay were also reported. A 12 m long flukeless T-24, with a diameter of 0.763 m, resulted in an average tip embedment of 1.7 L. Loads were applied in a horizontal direction, in view of the fact that such anchors were intended to only secure flexible risers, and ultimate capacities of 900 to 1100 kN were observed immediately following installation. Capacities were said to have increased by a factor of approximately two after ten day’s consolidation. Inclined 45° loading of a 620 kN (dry weight), 12 m long, 1.067 m diameter flukeless anchor, with an average tip embedment of 2.4L in normally consolidated clay resulted in ultimate capacities of 1900 to 2100 kN directly after installation. A setup factor of approximately two was observed after eighteen days consolidation. Vertical pullout tests of the same torpedo pile unconsolidated capacity of 800 kN, increasing by a factor of 2.5 – 2.75 following ten day’s consolidation.

Brandão et al. (2006) offer a fragment of information from full scale field tests at the Albacora Leste Field in the Campos Basin. Installation results, shown in Table 2.2,
correspond to a torpedo pile with a dry weight of 74 t. Unfortunately no anchor dimensions were given.

**Table 2.2 Torpedo Pile installation results offered by Brandão et al. (2006)**

<table>
<thead>
<tr>
<th></th>
<th>Test 1</th>
<th>Test 2</th>
<th>Test 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tip angle</td>
<td>60°</td>
<td>30°</td>
<td>30°</td>
</tr>
<tr>
<td>Water depth (m)</td>
<td>1195</td>
<td>1180</td>
<td>940</td>
</tr>
<tr>
<td>Release height (m)</td>
<td>40</td>
<td>135</td>
<td>97</td>
</tr>
<tr>
<td>Maximum velocity (m/s)</td>
<td>16.3</td>
<td>23.0</td>
<td>24.0</td>
</tr>
<tr>
<td>Embedment depth at padeye (m)</td>
<td>9.0</td>
<td>16.0</td>
<td>17.5</td>
</tr>
<tr>
<td>Final pile inclination</td>
<td>3.0°</td>
<td>9.0°</td>
<td>5.0°</td>
</tr>
</tbody>
</table>

In the second half of 2005, torpedo piles (T-98s featuring four flukes) were successfully employed for the first time to anchor a permanently moored facility; the P-50 FPSO vessel in the Albacora Leste Field (Ehlers et al. 2004; Santos et al. 2004; Brandão et al. 2006; Denney, 2007). The semi-taut leg spread-mooring system comprises 18 lines, each made of polyester rope and steel chain connected to a separate torpedo pile, with 10 lines to the bow of the FPSO and 8 lines to the stern. The 17 m long torpedo piles used comprise a shaft diameter of 1.07 m and four 0.9 m wide by 10 m long slender flukes. A maximum water depth of 1384 m was recorded at the installation sites. In a summary of the installation process, Denney (2007) notes a maximum tip embedment depth of ~ 2.2L and an average embedded pile inclination of less than 10°. For an instrumented torpedo pile featuring accelerometers and inclinometers, an impact velocity of 26.8 m/s was achieved 8.3 s after release into free-fall.

2.3.2.2 Physical modelling performance data

Laboratory tests at 1g have been conducted on ~ 305 mm long model Torpedo Piles of varying density in kaolin clay (Audibert et al. 2006; Gilbert et al. 2008). Release into free-fall above the mudline in air led to final tip embedment depths of 1 to 4.2L and subsequent anchor efficiencies ranging from 2 to 5. Embedment depths were shown to increase with drop height (and thus impact velocity) and pile mass however further installation insight is limited as prototype stresses and velocities cannot be replicated at 1g. The studies do reinforce the significant influence of post-installation consolidation time on the vertical capacity of dynamically installed anchors; indicating short-term
capacities of approximately 30% of the capacity at complete consolidation (see Figure 2.7a).

![Figure 2.7 (a) Dependence of Torpedo Pile capacity on time after installation (after Audibert et al. 2006); (b) Inclined load tests on zero fluke model Torpedo Piles (after Gilbert et al. 2008)](image)

Gilbert et al. (2008) report an obvious trend for increasing pullout capacity corresponding to a reduced loading angle to the horizontal at the pile padeye (see Figure 2.7b). An increase of approximately 34% in the ultimate capacity for 0° loading to the horizontal compared to purely vertical loading was observed. However, it is doubtful that purely horizontal loading at the padeye could have been achieved in the laboratory experiments without disturbing the soil in the vicinity of the piles. Nonetheless, the study did show that least resistance to uplift is through axial loading. This has also been confirmed in recent centrifuge modelling of flukeless Torpedo Piles (Liu et al. 2014). Here, it was shown that no matter which angle the projectile penetrated a soft soil sample, the smallest value of holding capacity ensues when it is extracted in the same line as its penetration orientation.

Hossain et al. (2014) reported interesting capacity findings from a series of centrifuge tests on two 1:200 reduced scale model dynamically installed anchors in a sample of lightly over consolidated kaolin clay and a sample of calcareous silt. Model ‘B’ corresponds to a Torpedo Pile while model ‘N’ relates more closely to a DPA (see Figure 2.8a). Impact velocities of 14.9 to 22 m/s were measured corresponding to embedment depths of 1.2 to 2L (see Figure 2.8b). The embedment was noted to be 1.6 times lower in the calcareous silt than in the clay for a similar impact velocity and attributed to the significantly higher shear strength of the silt; silt $s_u = 2 + 3z$ kPa and clay $s_u = 1 + 0.85z$ kPa. Unfortunately, in most instances of the clay penetration, the
final anchor embedment depths were restricted to the sample depth (~ 2L); implying that higher embeddings may have been achievable.

Figure 2.8 (a) 1:200 reduced scale dynamically installed anchors, (b) embedment depths achieved in the centrifuge tests (after Hossain et al. 2014)

The majority of data from the anchor pullout tests by Hossain et al. (2014) were presented as normalised capacity (similarly normalised to those described in section 2.3.1.2) against displacement of the anchor padeye normalised by the shaft diameter. Figure 2.9a shows that load response profiles in silt were more gradual and less brittle compared with those for clay. However, the displacements required for achieving maximum holding capacity, for a given inclination and post-installation consolidation time, were consistent. The anchor holding capacities in the clay sample are also seen to increase with increasing post installation consolidation time. Interestingly, the load displacement response profiles in the clay sample demonstrate a similar two peak characteristic to that of Richardson et al. (2009) discussed in section 2.3.1.2. Figure 2.9b shows that the anchor holding capacities increased with reduced loading angle to the mudline. As the loading angles reduced, capacity response curves became more gradual; indicating greater displacements being required to reach maximum holding capacities. The most brittle response in Figure 2.9b corresponds to a loading angle of 80°; i.e., it rises quickly to the peak value followed by a sharp drop. This is considered to be somewhat attributable to very little anchor rotation and (hence) less adherence of soil along the anchor sides. Interestingly, for similar periods of post-installation consolidation, the normalised holding capacity for Model ‘N’ was found to be 0.75–0.78 times that for Model ‘B’ regardless of loading angle.
Figure 2.9 Effect on load displacement response due to: (a) Soil type and post-installation consolidation time, (b) loading angle in kaolin clay sample (after Hossain et al. 2014)

2.3.3 **OMNI-Max™ anchor**

Zimmerman and Spikula (2005) proposed a new design for a dynamically installed anchor initially termed the *Self-Penetrating Embedment Attachment Rotating Anchor* or ‘SPEAR’. It was later referred to as the MIG Anchor (Zimmerman, 2006a) and finally the OMNI-Max Anchor (Shelton, 2007). Patents for the OMNI-Max anchor design and installation/recovery methodologies were issued in 2006 (Zimmerman, 2006b).

Somewhat similar to the DPA™ and Torpedo Pile concepts, the OMNI-Max™ is arrow shaped and initially self-penetrates the seabed through the kinetic energy gained during free-fall. Unique features of the device however include (a) a 360° rotating loading arm positioned along the anchor shaft to which the mooring line is attached and (b) retractable flukes (Figure 2.10). The hinged loading arm is designed to rotate about the anchor shaft toward the angle of load application to ensure that the largest projected anchor area is normal to the loading direction, hence maximising capacity. With an anchor uplift angle of 40°, the load is in-line with the arm while the anchor becomes more perpendicular to the load (generating increased bearing capacity).

The retractable flukes permit anchor adjustment to be made prior to deployment and can be set to expose increased/reduced surface area. The three fluke arrangement either side of the loading arm is a notable trait of the OMNI-Max anchor, varying from the four fluked DPA™ and Torpedo Piles. Larger flukes at the rear encompass a greater surface area than those near the tip. This design method aims to achieve an equilibrium resistance profile along the anchor considering typical deep-sea strength gradients which increase with depth.
Figure 2.10 OMNI-Max anchor including the loading sequence for (a) normal operating conditions (b) anchor rotation phase (c) further penetration phase during storm event, after (Delmar Systems, Inc., 2011)

Under normal operating conditions, the anchor is said to generate sufficient uplift capacity in its initial vertically embedded position following dynamic penetration. Upon application of a load greater than approximately 20% of its design capacity however, the anchor rotates in the soil until the lateral resistance of the lower flukes becomes equal to the lateral resistance of the upper flukes (Zimmerman et al. 2009). When this loading is surpassed, the anchor performance is governed by its axial capacity. In cases of extreme loading, the anchor will simply dive deeper into stronger soil at an altered trajectory until the resistance to further penetration is equal to the load being applied. This sequence is illustrated in Figure 2.10.

A key advantage of the OMNI-Max™ as outlined by (Shelton, 2007) is the non-catastrophic failure design. The mooring is to be limited by the mooring line rather than the anchor capacity. During a failure event, it is envisaged for the mooring line to break at a fairlead prior to realising ultimate anchor capacity. In such an incidence where the floating rig loses station keeping, the rig will not drag anchors across seafloor assets. Instead, the remaining anchors’ initial behaviour is to rotate, become more
perpendicular with the load and penetrate deeper. The additional required capacity would thus be collectively shared by the remaining mooring lines.

2.3.3.1 Field experience

Full-scale prototype tests of the OMNI-Max\textsuperscript{TM} took place in March 2007 at Green Canyon in the Gulf of Mexico; in water depths of \( \sim 1,650 \) m. The tests comprised one dynamic installation test followed by two capacity tests at varied loading angles on the same embedded anchor; 9.7 m long with a dry weight of 38 t. A limited summary of the main findings is given by Shelton (2007). A drop height of \( \sim 76 \) m was employed in an effort to realise the highest velocity at impact and a horizontal offset at the mudline from initial release positioning of \( \sim 6 \) m was observed. Tip penetration of approximately 1.23L was achieved directly following dynamic installation with an estimated angle of less than 10° from vertical reported. Both pull tests are said to have withstood 100% of the available winch power aboard the anchoring handling vessel for \( \sim 10 \) minutes (limited by over-heating issues with the vessel engines).

Zimmerman et al. (2009) report a total of 54 OMNI-Max\textsuperscript{TM} installations for temporary MODU moorings using an anchor with the dimensions and properties given in Table 2.3.

Tip embedment depths of 1.17 to 2.2L tip were observed over the entire installation database and it was said that all embedment orientations were “within tolerances”. The authors offer a brief account of the anchor performance during storm conditions (Hurricane Gustav) in the Gulf of Mexico in 2008. A MODU secured by an eight-leg OMNI-Max\textsuperscript{TM} mooring system was subjected to maximum 1-minute wind speeds greater than 100 knots. Following the hurricane, the MODU was found 3,700 m northwest of its original location attached to only one anchor. Post analysis suggests that individual anchor loads could have reached up to 5,477 kN (over 14 times anchor dry weight), based on the assumption that the angle between the anchor axis and applied load was \( \sim 45° \). Anchor tip embedments doubled from their original depths achieved through dynamic penetration, in some cases, with a maximum tip embedment of 4L observed post hurricane.

By 2011, it was reported that nearly 160 OMNI-Max\textsuperscript{TM} anchors had been installed across the Gulf of Mexico (Shelton et al. 2011).


2.3.3.2 Physical modelling performance data

Cenac Π (2011) conducted tests on a 1:24 reduced scale model OMNI-Max anchor and reported an average embedment depth of 1.5L in an artificial mud mixture following free-fall in water. Embedment depths were also found to increase with impact velocity (Figure 2.11a).

A study analysing the hydrodynamic properties of OMNI-Max anchors in water using 1:24 and 1:15 reduced scale models was carried out by Nie and Shelton (2011). Although soil installation and capacity test data were beyond the scope of the study, the authors did provide field embedment depth results which supplement those given by Zimmerman et al. (2009) (see Figure 2.11b).
Gaudin et al. (2013) detail a centrifuge study using a reduced scale OMNI-Max anchor (see Figure 2.12a) in an overconsolidated kaolin clay \( (s_u = 3 + 1.1z) \) and a calcareous silt \( (s_u = 3.3z) \). Impact velocities of 10.4 to 23 m/s led to tip embedment depths of 1.18 to 2L in the clay sample, within the range of field data reported by Zimmerman et al. (2009) for clay with a similar strength. Significantly lower embedment depths of 1.14 to 1.46L were achieved in the silt despite higher impact velocities of 20.5 to 29.4 m/s (see Figure 2.12b); due mainly to the higher undrained shear strength of the calcareous silt. The data corresponding to the silt sample indicate an increase in embedment depth with impact velocity. The scatter is greater for the tests in clay, however it was noted that these tests may have been affected by the presence of an anchor chain during penetration.

Figure 2.11 Dependence of OMNI-Max embedment depth on impact velocity (a) after Cenac II et al. (2011); (b) after Nie and Shelton (2011)
Monotonic load displacement response tests on the anchor initially installed to a tip embedment depth of 1.4L are shown in Figure 2.13a. Three profiles exhibit a peak load (corresponding to the anchor rotation and achievement of the maximum axial capacity at that embedment depth) before dropping and increasing again steadily with displacement (corresponding to the anchor diving mechanism). These profiles all correspond to load inclinations of zero degrees to the mudline. The remaining profile (‘SSP3’) corresponds to a higher load inclination of 15.3 degrees to the mudline and exhibits a continuous decrease in capacity after the keying peak. This is because the anchor was pulled towards the surface suggesting that there is an embedment depth limit for the diving mechanism to initiate. It should be noted that the very sharp drops in load along the profiles (and subsequent significant increase) are simply due to loading pauses where the centrifuge was ramped down momentarily to assess the padeye embedment depth (shown in Figure 2.13b). The embedment depth data indicate a
reduction of the padeye embedment during the initial keying phase. Manual probing during centrifuge ramp down periods indicated that the anchor had an inclination of approximately 30 degrees to the horizontal once the peak load had been reached.

Figure 2.13 Drag displacement response for OMNI-Max anchor: (a) with load; (b) with padeye embedment depth (after Gaudin et al. 2013)
2.3.4 Dynamically Embedded Plate Anchor (DEPLA)

The DEPLA is a relatively new dynamically installed anchor concept intended for mooring offshore energy facilities at deep-sea. It is described as a hybrid anchor system which combines the geotechnical efficiency of conventional plate anchors with the low installation costs associated with dynamically installed anchors. Its geometry is akin to that of the DPA™ and Torpedo Pile, featuring a central cylindrical shaft that may be fully or partially solid and a set of four flukes arranged on a cylindrical sleeve (Figure 2.14). Blake et al. (2014) propose a full scale DEPLA size of length 9 m and shaft diameter 0.72 m with a total dry mass (follower and flukes) of 37 t, suitable for mooring mobile offshore drilling units. Following dynamic penetration, the removable shaft is retrieved for use in further installations, leaving the anchor flukes vertically embedded in the seabed (Blake and O’Loughlin 2012). The flukes constitute the load bearing element as a plate anchor. When sufficient load develops in the mooring line, the plate rotates to an orientation that is approximately perpendicular to the direction of loading at the padeye, allowing full bearing resistance of the plate to be mobilized. The installation and keying processes are illustrated in Figure 2.14.

![Figure 2.14](image_url)

**Figure 2.14** Dynamically Embedded Plate Anchor (DEPLA) after Blake et al. (2014) including shaft removal and fluke keying process: 1. installation through self-weight and kinetic energy obtained during free-fall; 2. shear pin connecting follower and flukes parts, follower recovered for reuse, leaving flukes embedded in soil; 3. flukes keyed into position; 4. final orientation under load after Blake and O’Loughlin (2012)
2.3.4.1 Field experience

The DEPLA concept is in its infancy however field testing with models of reduced scale 1:12, 1:7.2 and 1:4.5 have been carried out in recent times (Blake and O’Loughlin 2012; O’Loughlin et al. 2013; Blake et al. 2014). Results indicate that the DEPLA exhibits similar behaviour to other dynamically installed anchors during installation, but the plate will offer much higher uplift efficiencies when loaded.

Shaft tip embedment depths of 1.4 – 3.7L were reported in soft soils following dynamic penetrations from a range of drop heights in water and some “jacked-in” penetrations. The extraction resistance of the removable shaft segment ranged from 2.1 to 4.7 times its dry mass reflecting an increase in soil shear strength with depth. These capacity ratios should be lower than other dynamically installed anchor as the DEPLA plate sleeve is expected to reduce the frictional surface area of the follower by between 30% and 50% depending on the model anchor configuration.

During loading of the plate anchor, the mooring line (connected to a single fluke as shown in Figure 2.14) often had a 90° angle of inclination to the mudline modelling a vertical mooring system. The plate keying and pullout response was characterised by an initial stiff response (as the anchor initiated rotation) followed by a softer response as the rotation angle increased, and a final stiff response towards a peak load as the effective eccentricity of the padeye reduced and the plate anchor capacity was fully mobilised (see Figure 2.15a). These stages become progressively less distinct as the load inclination to the horizontal reduces (see Figure 2.15b). For all load inclinations, the end of plate keying was shown to coincide with the ultimate capacity. The keying and ultimate capacity mobilisation phase resulted in a non-recoverable loss in embedment of 1 – 1.8 times the full plate diameter.

The ultimate vertical capacities of the plate varied from 6 to 42 times the plate anchor dry weight (see Figure 2.16) depending on final embedment depths achieved and hence impact velocity for the dynamic installations. As indicated by Figure 2.15b, the ultimate vertical capacities reduce with load inclination to the horizontal which is to be expected for plate anchors (e.g. Song et al. 2008; Yu et al. 2011). Interestingly, loading results for dynamically installed and “jacked-in” DEPLAs were similar, indicating that the rate of installation did not affect either the ultimate capacity or the plate keying response.
Figure 2.15 Load-displacement response of DEPLA plate during keying and pullout in the field: (a) Vertical monotonic loading (b) Inclined monotonic loading of a 1:12 scale anchor (after Blake et al. 2014)

Figure 2.16 Dependence on DEPLA capacity on final embedment depth achieved (after Blake and O’Loughlin 2012; Blake et al. 2014)

2.3.4.2 Physical modelling performance data

Centrifuge studies have typically supported the findings gathered from field data. O’Loughlin et al. (2013; 2014) give follower tip embedments in the range 1.9–2.1L for tests on a 76 mm long model DEPLA with impact velocities in the range 24 to 30 m/s. Tip embedments of 1.6 and 2.8L were also achieved by a 51.5 mm and 101.5 mm long model anchor respectively. Ultimate vertical capacities of the removable shaft segment ranged from 2.2 to 3.2 times the dry weight of the follower, with capacities increasing with follower length and hence frictional surface area. Interestingly, the loss in plate anchor embedment during keying in centrifuge tests is reported to be in the range 0.50–
0.68 times the plate diameter (much lower than the range 1–1.8 given for field tests). Furthermore, the soft response as the rotation angle of the plate increases during keying was noticeably different in centrifuge tests than in the field (Figure 2.17; stage ‘2’). The load here plateaus and then reduces slightly which may be attributed to local softening of the soil during plate rotation. Blake et al. (2010) consider the influence of post-installation consolidation time on the vertical capacity of the plates and show the ratio of short to long term capacity to be much higher than that of other dynamically installed anchors.

Figure 2.17 Load-displacement response of DEPLA plate during keying and pullout in the centrifuge for vertical monotonic loading (after O’Loughlin et al. 2014)

### 2.3.5 Other concepts/designs

The fundamental advantages of dynamically installed anchors in the offshore energy industry continue to inspire new anchor designs. Developers persist to find the optimum design concept. Muehlnur (2008) proposed an interesting dynamically installed anchor design which, like the DEPLA concept, aims to capitalise on the high holding capacities achieved by plate anchors. Following seabed penetration, loading of the anchor line causes the deployment of initially folded flukes. In their deployed positions, one or more flukes extend perpendicularly toward the anchor shaft, and function similarly to conventional plate anchors (Figure 2.18a).

Since the proposal of the Torpedo Pile, its inventors have considered modifications to the original design. Meideros (2011) describes a pile with enhanced clamping strength for vertical loading with the addition of at least two articulated movable plates with flaps. The flaps induce rotation of the plates when a vertical tensile force is exerted on
the pile until the plates stop at a fully extended position. The process, illustrated in Figure 2.18b, is somewhat similar to the Muehlner (2008) concept.

Centrifuge testing on three distinct dynamically installed anchors with equivalent total surface areas but varying geometries was reported by Brum Jr. et al. (2011). Each model comprised four flukes, equally spaced at 90° as shown in Figure 2.18c. Following quasi-static installations (0.5 mm/s) into a predominantly meta-kaolin clay sample to the same tip embedment depth of 2L, vertical loading led to anchor efficiencies of 2.4 to 2.7. Interestingly, the tests suggested that a Torpedo Pile design offered the lowest resistance to uplift and could be enhanced by up to 12% with the modified design for anchor 3.

Figure 2.18 Other dynamically installed anchor designs: (a) Folding dynamically installed anchor after Muehlner (2008); (b) Torpedo Pile with enhanced clamping strength concept after Medeiros (2011); (c) Model dynamically installed anchors with varying geometries after Brum Jr. et al. (2011).
2.4 Embedment prediction methods

Dynamically installed anchor capacities are strongly dependent upon the embedment depth achieved during installation. This is understandable considering that deep sea soil strengths typically increase with depth. Accurate capacity calculations are therefore dictated, to a large extent, by the success of predicting final embedment depth for a given anchor drop height and corresponding impact velocity.

Empirical equations and related analytical techniques to predict the depth of projectile penetration into natural earth materials were initially derived as a direct result of research conducted to evaluate the feasibility of a nuclear earth penetrating weapon in the early 1960s. Published by Young (1969), the rudimentary equations were based exclusively on fitting data acquired through full scale earth penetration tests with an accuracy of approximately ±15%. Revised equations have since been published to include new target materials (Young 1981; 1997). As part of its radioactive waste management research programme, the UK Department of the Environment commissioned a penetrometer study to assess the feasibility for final disposal of waste in deep-sea ocean bed sites. Embedment depths achieved by the free-fall penetrometers/waste canisters were of significant importance with a minimum of 10 m tail embedment and a 500 year barrier containment life considered as basic criteria. Equations were thus developed to help predict the embedment depths of 3 - 20 m long penetrometers with dry weights of 2 - 100 t, featuring three stabiliser flukes (Ove Arup and Partners 1982; 1983).

2.4.1 Analytical embedment model

In the beginning of the 1970s, the U.S. Naval Civil Engineering Laboratory (NCEL) launched a research programme to develop the capabilities of existing penetration evaluation equipment and assess a series of proposed penetration prediction techniques. Schmid (1969) and Migliore and Lee (1971) provided early versions of embedment equations for dynamic free-falling objects. A view to establishing an innovative anchoring technique in marine sediments was also desired, with recognition of accurate depth predictions being vital for determining direct embedment anchor performance (True, 1974). Early equations were modified by True (1976) to accommodate for higher projectile velocities. By this time it had been established that resistances encountered during dynamic penetration were higher than values predicted using static formulae. True proposed that the combined effects of a soil strain rate effect factor, an interface
friction ratio and a drag resistance force could be applied to conventional static formulae to describe the elevated dynamic penetration resistances satisfactorily. The theoretical embedment model developed by True (1976) was published in the US Handbook for Marine Geotechnical Engineering (Rocker, 1985) and has not been significantly changed for inclusion in the second edition of the handbook (Thompson et al. 2012).

The method is deemed applicable for long slender projectiles impacting cohesive sediments at velocities greater than ~ 0.9 m/s; dynamic effects at lower velocities being considered negligible. An iterative procedure, whereby all forces acting on the projectile are calculated on the basis of mudline impact velocity and the soil properties, is employed over equal finite increments. This enables best interpretation of the velocity dependent resistance forces. The difference between the weight of the projectile (W), plus any external driving force (if any), and the combined soil resistance forces is given as being equivalent to the net downward force (F) acting on the projectile:

\[ F = W - R_f(F_{\text{bear}} + F_{\text{frict}}) - F_d - F_b \]  

where \( F_{\text{bear}} \) is the bearing resistance, \( F_{\text{frict}} \) is the side friction resistance, \( R_f \) is a strain rate factor, \( F_d \) is the drag resistance and \( F_b \) is the buoyancy force simply given as the weight of the displaced soil. These forces are illustrated in Figure 2.19 and Figure 2.20 (acting on a flukeless penetrometer and a 4 fluke dynamically installed anchor respectively) and are discussed in greater detail in the following sections.

Figure 2.19 Forces acting on a penetrator before and after mudline impact (after Thompson et al., 2012)

Figure 2.20 Resistance forces acting on a dynamically installed anchor during installation
2.4.2 Bearing, frictional and buoyancy forces

The bearing and frictional resistances are given by:

\[ F_{\text{bear}} = s_u N_c A_p \]  \hspace{1cm} 2.2

\[ F_{\text{frict}} = \alpha s_u A_s \]  \hspace{1cm} 2.3

where \( s_u \) is the undrained shear strength averaged over the appropriate side area, \( A_s \), or projected area, \( A_p \), \( N_c \) is a bearing capacity factor and \( \alpha \) is an interface friction ratio (of limiting shear stress to undrained shear strength). During dynamic penetration of projectiles in soil, the interface friction ratio may be conveniently estimated as the inverse of the soil sensitivity as a first order approximation. Although this approach essentially characterises soil-to-soil friction (Andersen et al. 2005), numerous relevant past studies have successfully applied it for quantifying the frictional resistance generated between the steel wall of a foundation and the soil.

A constant \( N_c \) value is often adopted for the projectile tip such as \( N_c = 12 \) (Beard 1977, O’Loughlin et al. 2004a, Richardson 2008, O’Loughlin et al. 2009, O’Loughlin et al. 2013b), 15 (Freeman and Schuttenhelm 1990), 10 (Mulhearn et al. 1998), 17 (Gilbert et al. 2008) and 14 (Steiner et al. 2012, Steiner et al. 2014). \( N_c = 7.5 \) is typically adopted for the upper and lower end of dynamically installed anchor flukes (Richardson 2008, O’Loughlin et al. 2013b) based on a solution for deeply embedded strip footings (Skempton 1951). Numerical studies often describe \( N_c \) as a function of embedment ratio (e.g. Aubeny and Shi 2006, Abelev et al. 2009) or as a function of the rigidity index in addition to the embedment ratio (e.g. Nazem et al. 2012).

During free-fall through the water column, the hydrodynamic resistance forces exerted on a projectile are buoyancy and drag (drag effects are discussed in the following section). Buoyancy (\( F_b \)) is simply the weight of the displaced water, given as:

\[ F_b = \rho_w g V_{\text{total}} \]  \hspace{1cm} 2.4

where \( \rho_w \) is the water density and \( V_{\text{total}} \) is the total volume of the immersed projectile. As the projectile penetrates the mudline, the shifting density from water to that of the soil will need to be accounted for. This is usually done through an iterative calculation process over displacement intervals until the projectile is fully submerged in the sediment and the soil density (\( \rho_s \)) is used in equation 2.4.
2.4.3 Drag effects

The total drag force acting against the direction of motion of a projectile in a fluid consists of two components: the form or pressure drag force ($F_{d,p}$) and the viscous or frictional drag force ($F_{d,s}$). Pressure drag, caused by an adverse pressure gradient between the front and rear of the object (leading to flow separation), comprises the integrated pressure acting on the frontal or projected area, $A_p$, while frictional drag is the sum of shear stresses along the projectile boundary or surface area, $A_s$.

For most practical cases the contribution of friction drag to total drag is very low; Achenbach (1971) reported ~ 2 to 3% for spheres while more recently Raie (2009) identified that only 12% of the total resisting force acting on Torpedo Anchors at soil impact was due to friction drag. Consequently, most dynamically installed anchor studies combine the pressure and frictional drag forces into one governing equation:

$$F_d = \frac{1}{2} C_D \rho_w A_p v^2$$

where $F_d$ is the total drag force and $C_D$ is a total drag coefficient dependent on the projectile geometry and surface roughness. Although $A_p$ is shown as the reference area in equation 2.5, some dynamic projectile studies employ $A_s$ (e.g. Shelton et al. 2011).

It is important to note however, that frictional drag can be dominant at very low flow rates and hence low Reynolds number ($Re$). For Newtonian fluids where the shear stress is linearly proportional to the shear strain rate (e.g. air and water), $Re$ is defined as the ratio of the inertial forces to the viscous forces, expressed as:

$$Re = \frac{\nu D}{v} \text{ where } v = \frac{\eta}{\rho}$$

where $D$ is the projectile diameter, $v$ is the velocity of the object relative to the fluid, $v$ is the kinematic viscosity of the fluid, $\eta$ is the absolute viscosity and $\rho$ is the fluid density.

The increase in frictional drag is best understood by considering the simplified geometry of a two-dimensional circular cylinder for which the experimental correlation of $C_D$ against $Re$ has been investigated extensively over a wide range of flow rates. As $Re$ reduces below a critical value (~ $3 \times 10^5$) the downstream wake configuration and flow separation between the fluid and the cylinder changes significantly as depicted in Figure 2.21a. These changes result in very high $C_D$ values (as seen in Figure 2.21b) particularly below $Re \sim 2 \times 10^5$ where frictional drag dominates.
Figure 2.21 Influence of Reynolds number for a circular cylinder on: (a) the downstream wake configuration (after NPTEL 2015) and (b) the drag coefficient (after Brown and Lawler 2003)
2.4.3.1 Relevant projectile drag coefficients

Most dynamically installed anchor studies assume a constant $C_D$ in both water and soil regardless of variations in Re. True (1974) specifies $C_D = 0.7$ as a useful average approximation for a slender cylindrical object. More recent studies have recommended a value of 0.65 for the OMNI-Max anchor (Shelton 2007) and 0.63 for an ellipsoidal tipped DPA™ (Oye 2000 and Richardson 2008). Discrepancies are to be expected however when considering that $C_D$ depends on the projectile geometry and Re. Freeman et al. (1984) for instance proposed $C_D$ values of 0.15 to 0.18 for a 1.8 t long slender cylindrical projectile for velocities of 10 to 50 m/s. A study on seven density modifiable Torpedo Piles with varying aspect and scale ratios, and fluke magnitudes (Hasanloo et al. 2012) specified $C_D$ values of 0.2 to 1.2 relating to variations in Re of between $4.8 \times 10^5$ and $2.16 \times 10^6$ (see Figure 2.22a). A further study on OMNI-Max anchors using 1:15 reduced scale models (Cenac Π 2011) indicated drag coefficients of $C_D = 0.46$ to 0.83 and $C_D = 0.7$ to 1.12 from free-fall and tow tank tests respectively with varying Re (see Figure 2.22b). Figure 2.22 shows that for a given projectile geometry at a specific Re between $10^5$ and $10^6$, the $C_D$ takes an abrupt dip. This phenomenon is referred to as the drag crisis and is due to the reduction of the downstream turbulent wake as the laminar-separated flow transitions to turbulent flow (refer to Figure 2.21a).

Shelton et al. (2011) utilise the International Towing Tank Conference equations (ITTC 1957, 2008) to calculate frictional drag coefficients in an effort to incorporate varying Re:

$$C_{D,s} = \frac{0.075}{(\log_{10} Re - 2)^2}$$  \hspace{1cm} (2.7)

However, equation 2.7 was specifically denoted for model-ship correlations. It was not meant to represent the resistance of plane or curved surfaces, nor was it intended to be used for such a purpose as specified in the ITTCs.

As with the buoyancy calculations, care must be taken when calculating the drag resistance during the transition from water to soil to account for the changing medium density. It is appropriate to note that there will also be a change in the Re during this transition phase, implying separate drag coefficients for projectile motion through water and through soil. However a constant drag coefficient is typically assumed for dynamically installed anchor studies. This is reasonable provided the change in
Reynolds number is small, considering that the projectile drag resistance accounts for only a small portion of the total soil penetration resistance.

Figure 2.22 Drag coefficient variation with Reynolds number for (a) Torpedo Pile (after Hasanloo et al. 2012); (b) OMNI-Max (after Cenac II 2011)
2.4.3.2 Trailing line drag effects

A trailing mooring line is attached to all dynamically installed anchors during installation. It is crucial to consider the effects it may have on the projectile velocity and vertical orientation during both free-fall through water and soil penetration. For projectiles with no trailing line, a terminal velocity would be reached during free-fall through water (provided a sufficiently high drop-height above the seabed is allowed for) when the following equation is balanced:

\[ mg = F_d + F_b \]  

where \( m \) is the total mass of the projectile and \( g \) is the acceleration due to gravity.

Early feasibility studies on the DPA™ using reduced scale models (1:25) in a water-filled tank were carried out to analyse the effects of the trailing anchor line during free-fall and dynamic penetration (Lieng et al. 2000). The tests assessed the influence of varying vertical lengths of line (a) and chain loop widths (c) as denoted in Figure 2.23. Detailed results have not been published however the following was reported (Deep Sea Anchors 2009):

1. The trailing line does not alter the behaviour of the anchor during free-fall other than “gently tugging vertically” at the padeye connection due to hydrodynamic drag.
2. Less drag is realised with increasing line length suspended above the anchor.
3. Line drag does not “noticeably” alter anchor velocity.

However, full scale field tests of the Torpedo Pile in the Campos Basin revealed that initially predicted penetration depths were not achievable due to the drag effects on a polyester mooring line, which reduced the projectiles’ ultimate velocities (Brandão et al. 2006). A steel wire rope and modified quick release mechanism was subsequently used in an effort to reduce the effects of drag. Nevertheless, the authors asserted that a shooting height limit above the seabed does exist at which a velocity threshold is reached. Beyond this threshold the anchor’s velocity starts to decrease. This is portrayed in the results of three varying drop heights using a 74 t anchor. A 97 m drop height resulted in an embedment depth of 17.5 m while a 40 m and a 135 m drop height realised 51% and 91% of this embedment depth respectively.
Furthermore, the drag coefficient of the DEPLA has been shown to reduce with anchor scale in field tests reported by O’Loughlin et al. (2013). This reflects the lessening effect that a 12 mm diameter rope has on anchors of increasing scale. A 1:7.5 scale anchor exhibited a maximum velocity of 13.4 m/s after a free-fall distance of about 30 m; beyond this distance the velocity was said to reduce. Finally, Cenac II (2011) reported that the mooring line was found to increase the drag resistance by up to 13.5% for an OMNI-Max anchor.

Equation 2.8 should therefore be modified to include the drag effects of a trailing line \( F_{d,l} \) such that:

\[
m g = F_d + F_b + F_{d,l}
\]
2.4.4 Strain rate effects

It has long been recognised that the undrained shear strength of cohesive soils can increase with the rate of straining (Casagrande and Wilson, 1951). The strength increase may be as high as a factor of 5 when soil shears in response to a rapidly penetrating object (Migliore and Lee 1971, Dayal et al. 1975). This phenomenon, termed the strain rate effect, often introduces uncertainties in predicting the mobilised or operational shear strength of soils \( (s_{u,mob}) \) during dynamic penetration. At very high shear strain rates, viscous effects dominate the undrained shear strength of soil and the shear strength increases with increasing shear strain rate (Sheahan et al. 1996, Lunne and Anderson 2007, Jeong et al. 2009). An example of the effect of strain rate on penetration resistance of T-bar and ball penetrometers in kaolin clay is shown in Figure 2.24, where the strain rate is approximated by \( v/d \) (\( v \) is penetration velocity and \( d \) is penetrometer diameter) and the penetration resistances \( q_{T-bar} \) and \( q_{ball} \) (for the T-bar and ball respectively) are normalised by the vertical effective stress, \( \sigma'_v \).

![Figure 2.24 Rate dependence of resistance ratios at large (undrained) strain rates (after Lehane et al. 2009)](image)

Several models have been used to describe the undrained strength dependency on the rate of straining, including a semi-logarithmic law proposed by Mitchell (1993) which expanded on an earlier version given by Graham et al. (1983):
an inverse hyperbolic sine law (Mitchell 1993):

\[ s_{u,mob} = s_{u,ref} \left[ 1 + \lambda \log \left( \frac{\dot{\gamma}}{\dot{\gamma}_{ref}} \right) \right] \tag{2.10} \]

and a power law (Biscontin and Pestana 2001):

\[ s_{u,mob} = s_{u,ref} \left( \frac{\dot{\gamma}}{\dot{\gamma}_{ref}} \right)^\beta \tag{2.11} \]

where \( s_{u,mob} \) is the mobilised undrained shear strength at the current strain rate \( \dot{\gamma} \), \( s_{u,ref} \) is the undrained shear strength at some reference strain rate \( \dot{\gamma}_{ref} \) and \( \lambda \), \( \lambda' \) and \( \beta \) are the strain rate parameters. The strain rate factor, \( R_f \), is thus given as:

\[ R_f = \frac{s_{u,mob}}{s_{u,ref}} \tag{2.13} \]

The semi-logarithmic law has been the most commonly adopted model for quasi-static strain rate studies in the past. Graham et al. (1983) prescribe an approximate 5 - 20% increase in shear strength for each order of magnitude increase in the rate of shear strain. This has been supported by Randolph (2004) who suggests \( \lambda \) parameter values of 5 - 20% per log cycle of shearing rate. Sheehan et al. (1996) reported \( \lambda \) to be up to 17% for strain rates ranging from 0.0014 - 670 %/hr in triaxial compression tests. This has been supported by \( \lambda \) values of 1.6 - 20% reported by Chow and Airey (2013) for a triaxial rate study. Furthermore, work reported by Lehane et al. (2009) demonstrates a 14 and 18% increase in ball and T-bar penetrometer resistance respectively per log cycle increase at penetration velocities spanning five orders of magnitude. These findings, together with the semi-logarithmic model, are commonly used as a basis for recent numerical and physical modelling studies involving dynamic projectile installations (e.g. Aubeny and Shi 2006, Nazem et al. 2012, Morton and O’Loughlin 2012, Chow and Airey 2013).

The power law has been shown to exhibit better agreement over datasets which would span the range of strain rates characteristic of marine dynamic penetration events (Biscontin and Pestana 2001, Abelev and Valent 2009). Some \( \beta \) parameter values from relevant studies are given in Table 2.4.
2.4.4.1 Strain rate parameters at higher peripheral velocities

It is evident that some experimental results have a tendency of deviating significantly from the strain rate models at higher peripheral velocities. Abelev and Valent (2009) suggest a modified power law model following tests on a silty clay material over a rate range of 0.25 - 1000 rotations/min. The model (equation 2.12) introduces a material constant, b, and agreed well with the very high strain rates tested in the shear vane study. However, it has yet to be used in any dynamically installed anchor analyses.

\[ s_{u,\text{mob}} = s_{u,\text{ref}} \left[ 1 + b \left( \frac{\dot{\gamma}}{\dot{\gamma}_{\text{ref}}} \right)^{\beta} \right] \]  

2.14

Back calculated λ and β strain rate parameter values derived from a suite of centrifuge studies on free-falling projectiles (Richardson et al. 2006, 2009; Richardson 2008; O’Loughlin et al. 2004, 2009) were reported by O’Loughlin et al. (2013). Figure 2.25a shows that the ranges of λ and β are typically 0.2 to 1.0 and 0.06 to 0.17 respectively as the average anchor velocity during penetration divided by the effective projectile diameter \((v_{av}/d_{eff})\) increases from 500 to 4250 \(s^{-1}\) (corresponding to impact velocities of 0 to ~ 30 m/s). Gaudin et al. (2013) also report high back-calculated strain rate parameters from centrifuge tests on OMNI-Max anchors in overconsolidated kaolin clay and calcareous silt of \(\beta = 0.16\) and 0.19 respectively (Figure 2.25b).

This wide variation in the strain rate parameter values may be (at least in part) attributed to the order of magnitude difference between the strain rate, \(\dot{\gamma}\) and the reference strain rate, \(\dot{\gamma}_{\text{ref}}\). The maximum \(\dot{\gamma}\) associated with the free-falling experiments in the centrifuge can be as high as 4250 \(s^{-1}\) or up to four to five orders of magnitude greater than \(\dot{\gamma}_{\text{ref}}\) for a standard penetration rate of 0.2 \(s^{-1}\) and up to 10 orders of magnitude greater than the nominal strain rate associated with laboratory triaxial compression tests and shear vane...
tests which lie in the range $3 \times 10^{-6}$ to $2 \times 10^{-3}$ s$^{-1}$ (Einav and Randolph 2006). Therefore it may be inappropriate to deduce strain rate parameters from laboratory analysis to be adopted in free-fall experiments in the centrifuge or the field. Similarly, caution must be taken when comparing the strain rates in the centrifuge to field tests. The strain rates (proportional to $v/D$) in the centrifuge are higher because absolute field and centrifuge impact velocities are comparable but the diameter is scaled in centrifuge tests.
2.4.4.2 Influence of soil stress history

Literature regarding strain rate dependence on the stress history of a soil sample is conflicting. Results reported by Lehane et al. (2009) for penetration tests in soil samples with overconsolidation ratios (OCRs) of 1, 2 and 5 indicate an increase in rate dependence with increasing OCR. Balderas Meca (2004) however reported reduced rate effects for increasing OCRs. Moreover, other studies suggest that the strain rate is virtually independent of the OCR (e.g. Graham et al. 1983; Lunne and Anderson 2007). This conflicting literature indicates a need for more experimental research to be carried out over a range of OCRs and at strain rates spanning several orders of magnitude.

2.4.4.3 Separate frictional and bearing rate functions

It has also been acknowledged that rate effects acting on penetrometers could be higher for the shaft (frictional) resistance than for the tip (bearing) resistance (Dayal and Allen 1973; Steiner et al. 2014; Chow et al. 2014). This would necessitate a separate rate function to be used on the resistance components. It is probable that the higher rate effect for shaft resistance is due to the higher strain rate at the cylindrical shaft involving curved shear bands, which can be estimated to be 20-40 times $\dot{\gamma}$ using a rigorous energy approach maintaining equilibrium (Einav and Randolph 2006). Chow et
al. (2014) report the successful application of separate rate functions to free-fall piezocone data from centrifuge tests in soft clay using the modified power law model:

\[
R_f = \left( \frac{n \hat{\gamma}}{\hat{\gamma}_{ref}} \right)^\beta
\]

where \( n = 1 \) for tip resistance (Zhu and Randolph 2011) and \( n \) is a function of \( \beta \) (adopted from work presented in Einav and Randolph 2006) for shaft resistance such that:

\[
n = 2 \left( \frac{n_1}{\beta} + n_1 - 2 \right)
\]

where \( n_1 \) is taken as 1 for axial loading.

However it is difficult to distinguish the rate dependence of each resistance component for dynamic penetrations and studies typically use a single overall rate function.

### 2.4.4.4 Final discussion on strain rate

The nominal shear strain-rates of triaxial compression tests and in situ shear vane tests are approximately \( 3 \times 10^{-6} \, \text{s}^{-1} \) and \( 2 \times 10^{-3} \, \text{s}^{-1} \) respectively (Einav and Randolph 2006). These rates are four and seven orders of magnitude lower than the typical rate of \( 25 \, \text{s}^{-1} \) associated with dynamically installed anchors in the field. Therefore it is difficult to extrapolate strain-rate parameters from laboratory and in situ tests to be adopted in the predicting the embedment depth of dynamically installed anchors into the seabed.

### 2.4.5 Analytical embedment model application

As previously mentioned, the prediction model utilises an iterative process over equal depth increments to account for velocity and depth dependent resistances. The net downward force is related to the deceleration of the projectile and is calculated using Newton’s second law of motion (modified to eliminate the time parameter):

\[
F_i = m v_i \left( \frac{dv}{dz} \right)
\]

where \( m \) is the projectile mass and \( dv/dz \) is the instantaneous change in velocity with depth. The change in velocity over each increment is thus determined as:
\[ \Delta v_i = \frac{F_i \Delta z}{m v_i} \]  

2.18

The new velocity for the \((i+1)^{th}\) increment is given by:

\[ v_{i+1} = v_{i-1} + 2\Delta v_i \]  

2.19

In order to begin the incremental calculations, the velocity of the projectile at the end of the first penetration depth increment \((v_1)\), is estimated as:

\[ v_1 = v_0 + \left( \frac{1}{v_0} \right) \left[ \left( \frac{\Delta z}{m} \right) (W_{s,0.5} - F_{\text{bear,0.5}} - F_{\text{frict,0.5}} - F_{d,0.5}) \right] \]  

2.20

where \(v_0\) is the impact velocity of the projectile at the mudline and \(W_{s,0.5}\), \(F_{\text{bear,0.5}}\), \(F_{\text{frict,0.5}}\) and \(F_{d,0.5}\) are the submerged weight, bearing resistance, frictional resistance and drag resistances respectively at mid-depth over the first penetration increment.

This approximation can then be used to calculate the respective force values at the end of the first depth increment. Subsequent iterations are made by recalculating \(W_{s,i}\), \(F_{\text{bear,i}}\), \(F_{\text{frict,i}}\) and \(F_{d,i}\) and the procedure continues until a negative value for \(v_{i+1}\) is reached. The projectile penetration depth is then calculated by interpolating between the last two velocity magnitudes:

\[ z = z_i + \Delta z \left( \frac{v_i}{v_i - v_{i+1}} \right) \]  

2.21

Some shortcomings of the analytical embedment model described in this section have been outlined in several computational fluid dynamic modelling studies (e.g. Raie and Tassoulas 2009). The most noteworthy shortcomings include:

1. The strain-rate effect in the soil is only related to the velocity of the projectile while in principle it may also be affected by the projectile’s geometry.

2. The model is limited to providing the projectile acceleration/velocity profile during penetration and the final embedment depth. It does not supply any information regarding the state of stress in the surrounding soil or the distribution of pressure and shear along the projectile.

3. The water is not modelled so there is uncertainty in conserving momentum when the projectile impacts the soil and in calculating the drag resistance forces acting on the projectile and trailing line.
Nonetheless, the model developed by True (1976) remains the most extensively adopted in dynamically installed anchor studies (e.g. Lieng et al. 1999; Medeiros 2002; O’Loughlin et al. 2004, 2009, 2013; Audibert et al. 2006).

2.4.6 Total Energy approach

A robust method for comparing the final tip embedment depth trends for a wide range of past centrifuge and field studies on dynamically installed anchors in non-dimensional form was proposed by O’Loughlin et al. (2013). It was shown that comparisons can be simplified by considering the total energy, $E_{total}$, of the anchor as it impacts the soil while removing the influence of diameter and soil strength gradient. $E_{total}$ is defined as the sum of the kinetic and potential energy (relative to final embedment depth):

$$E_{total} = \frac{1}{2}mv_i^2 + m'gz_{e,tip}$$

where $m$ is the anchor dry mass, $v_i$ is the impact velocity, $m'$ is the submerged mass of the anchor in soil and $z_{e,tip}$ is the final embedment depth of the anchor tip. A conservative relationship was established to harmonise a very large dataset that encompassed a range of anchor masses, geometries and impact velocities, (see Figure 2.26) expressed as:

$$\frac{z_{e,tip}}{d_{eff}} \approx \left( \frac{E_{total}}{kd_{eff}^4} \right)^{1/3}$$

where $k$ is the undrained shear strength gradient with depth and $d_{eff}$ is the effective anchor diameter (which accounts for the additional projected area of the anchor flukes).
Figure 2.26 Total Energy approach showing comparisons over a centrifuge and field embedment data (after O’Loughlin et al. 2013)

The Total Energy approach removes the influence of soil shear strength and anchor diameter, and may be a useful predictive tool for comparing anchors with markedly different geometries/mass/impact velocity. However, the approach does not capture the motion of the anchor through the soil as the analytical embedment model does. Oversights may therefore exist on the complex mechanisms that occur during dynamic embedment. One such oversight is that the model does not account for variations in soil sensitivity; higher sensitivities leading to lower frictional resistances and higher embedment depths.
2.5 Hole closure

Little is known regarding the behaviour of undrained soil directly following the penetration of a dynamically installed projectile. As outlined in the previous sections, the most commonly adopted embedment prediction model utilises an iterative process whereby individual bearing and frictional resistances are calculated at equal time intervals throughout soil penetration. Once a projectile tip embedment of 1L is reached (i.e. it has become fully submerged in the soil), resistance forces acting at its rear must be accounted for in the model.

For an instantaneous fully closed cavity in the wake of the advancing projectile, \( F_{\text{bear}} \) in equation 2.1 should enlarge to include a reverse end bearing resistance force at the projectile’s rear. For a fully open cavity, reverse end bearing would be omitted and \( F_b \) should increase to include the combined volume of the embedded projectile and the cylindrical cavity that extends to the mudline (see Figure 2.19). These variations in calculations can significantly affect the theoretical final embedment depths albeit that the effects will lessen as the projectile aspect ratio increases. Unfortunately, rear resistance forces have been overlooked in many dynamic projectile embedment studies to date including Rocker (1985) and Thompson et al. (2012).

2.5.1 Quasi-static studies

Significant effort has been invested in understanding the stability of axisymmetric excavations in clays and more recently the hole closure behaviour behind quasi-static penetrometers and shallow foundations in clays. Two independent soil failure mechanisms are now recognised by the geotechnical community: wall failure and backflow failure.

2.5.1.1 Wall failure

Excavations are quite common in the construction industry (e.g. inspection or access chambers and excavations for bored piles or piers). Consequently, numerous studies regarding their base and wall stability in clay with uniform undrained shear strength have been carried out (e.g. Skempton, 1951; Meyerhof, 1972; Prater, 1977; Britto and Kusakabe, 1982). Britto and Kusakebe (1983) employ the dimensionless stability number (N) to compare those studies listed defined by:
where $\gamma$ is the soil bulk unit weight and $H_w$ is the height of the excavation. Through the comparisons (see Figure 2.27), the authors conclude that for undrained soils with uniform strength the most critical failure mode for unsupported excavation is wall failure; where the open cavity collapses and an inward and downward displacement of soil extends to the soil surface. For undrained soils featuring a strength gradient ($k$), where strength increases with depth, base failure is unlikely because failure would have to develop in a region of the soil where the shear strength is greater than the soil above it. Therefore, although few excavation stability studies have analysed soils featuring a strength gradient, it is justifiable to assert that wall failure is the most critical failure mode.

Figure 2.27 Stability number versus depth of excavation/radius of excavation for various studies on excavation stability (after Britto and Kusakabe, 1983)

To incorporate strength gradients and mudline strengths ($s_{u,0}$) into the stability number calculations, Britto and Kusakabe (1983) offer the updated stability number equation for cylindrical excavations:

$$N = \frac{\gamma H_w}{s_{u,0} + k H_w}$$

The critical depth of the exaction ($H_{cr}$), i.e. the depth to which the excavation will remain open, is defined as:
\[ H_{cr} = \frac{N s_{u,0}}{\gamma M} \quad \text{where} \quad M = \frac{s_{u,0}}{s_{u,0} + k H_w} \]  

2.26

This approach is given in SNAME (1997) to predict wall failure or collapse of a cavity wall, however a flow failure mechanism proposed as early as the beginning of the 1990s (Craig and Chua, 1991) is now recognised as the critical failure mode for quasi-static penetrometers in undrained soil.

2.5.1.2 Backflow failure

For nominally undrained rates of penetration, backflow failure refers to the progressive infilling of soil at the rear of a penetrometer as it moves deeper into the soil. Recent studies have clarified that the cavity depth due to backflow failure is much shallower than the criterion for wall failure. In fact, Hossain et al. (2005) reported that conventional wall failure calculations for estimating the point of backflow onto spudcans in uniform strength clays can overestimate the stable cavity depth by up to four times.

To understand the backflow failure mechanism it is helpful to consider three distinct phases of soil penetration (as illustrated for a spudcan in Figure 2.28):

1. Shallow penetration (surface failure); here the soil translates outwards and upwards to the mudline as heave and an open cavity above the penetrometer is formed.
2. Onset of backflow; upon reaching a certain penetration depth, the soil starts to flow onto the rear of the penetrometer and it becomes partially covered.
3. Deep penetration (fully localised failure); upon reaching a certain penetration depth known as the transition depth, the penetrometer becomes fully submerged due to the continued infilling caused by backflow.

Transition depths to define the point of hole closure due to backflow over quasi-static penetrometers are now commonly expressed as a function of the dimensionless strength ratio \( s_u/\gamma D \), where \( s_u \) is the undrained shear strength, generally identified at the depth at which the hole closes and \( D \) is the diameter of the hole. This technique has been employed for spudcans (Hossain et al. 2005), pipelines and T-bar penetrometers (White et al. 2010; Tho et al. 2012) and ball penetrometers (Zhou et al. 2013; Morton et al. 2014). These studies show that higher strength ratios are associated with higher
transitional depths between shallow and deep penetrations (see Figure 2.29), i.e. deeper stable cavities/open holes in the wake of the penetrometer.

![Figure 2.28 Digital images and corresponding soil flow vectors of spudcan penetration tests in overconsolidated clay clearly showing: (a) shallow penetration, (b) onset of backflow failure mechanism, (c) deep penetration (after Hossain et al. 2005)]](image)

![Figure 2.29 Summary illustration of higher strength ratios corresponding to higher transitional depths for spudcans, T-bar and ball penetrometers (after Morton et al. 2014)](image)
2.5.2 Dynamic studies

In a report outlining work conducted by Sandia National Laboratories, Burdett and Karnes (1986) proposed that the penetration of a streamlined projectile into soft fine-grained seafloor sediment will be followed almost immediately by hole closure and would prevent any further flow of water in behind the advancing penetrator. An approximate solution indicated that an impact velocity greater than 15 m/s would be sufficient to close the hole and that a higher impact velocity would simply close the hole faster.

Poorooshasb and James (1989) presented evidence from a series of centrifuge tests which suggested cavities were present in the wake of dynamic projectile installations. Despite observing closed entrance craters at the surface of the normally consolidated speswhite kaolin samples, radiographs revealed open pathways nearer the rear of the embedded projectiles. However, the 13 g, 60 mm long, 6 mm diameter, blunt nose projectiles had been artificially accelerated using an air gun to reach impact velocities of 40 m/s in an acceleration field of 100g. At prototype scale, the 13 t projectile could not reach such a velocity during free-fall through water, having a terminal velocity of approximately 17.8 m/s assuming a conventional $C_D = 0.7$. This raises concerns regarding the applicability of the study for free-fall projectiles. Einav et al. (2004) present numerical analysis of the complete penetration process of DPAs using both rate-dependent and rate-independent constitutive relations for the anchor-soil interface and soil medium. Immediately after the anchor comes to rest in the soil, negative excess pore pressures were predicted towards the rear of the anchor (see Figure 2.30a). These suctions were related to material flow closing the gap in the wake of the advancing anchor.

Aubeny and Shi (2006) made the “simplifying assumption that a cylindrical void forms” in the wake of an impact penetrometer with $L/D = 4.25$. However, the study provided no reasoning as to why such an assumption was made. Furthermore, the theoretical embedment model employed did not account for the drag forces that contribute to penetration resistance.
Figure 2.30 Changes in pore pressure in the vicinity of penetrometers following dynamic installation: (a) after Einav et al. (2004), (b) after Sabetamal et al. (2014)

When considering the penetration resistance forces acting on model DPAs in centrifuge tests, O’Loughlin (2013) proposed that instantaneous hole closure would only occur for cylindrical projectiles with an L/D ≤ 2. It was therefore considered that the cavity created by the passage of the DPA shaft would remain open during the time taken for the anchor to come to rest in the soil. The authors note that full shaft hole closure would eventually occur but not within the time taken for dynamic penetration. Prior to this
study, O’Loughlin et al. (2009) assumed “partial” hole closure would occur instantaneously behind the DPA shaft. Both studies also assume full instantaneous hole closure behind the anchor flukes owing to their relatively low thickness and apparent plane-strain conditions.

Gaudin et al. (2013) reported closed holes behind a model OMNI-Max anchor through visual inspections following penetration in calcareous silt \((s_u = 3.3z;\) resulting in a nearly undisturbed surface) and an over consolidated clay \((s_u = 3 + 1.1z;\) resulting in a significant crater at the anchor entry point). Hossain et al. (2014) on the other hand found that the cavity formed by dynamically installed anchor models remained open in a sample of calcareous silt \((s_u = 2 + 3z\ \text{kPa})\) but was closed in a lightly over consolidated clay \((s_u = 3 + 1.1z)\) (see Figure 2.31).

![Figure 2.31](image-url)

**Figure 2.31** Cavity observations at the mudline after installing model anchors: (a) kaolin clay; (b) calcareous silt (after Hossain et al. 2014)

### 2.5.3 Potential effects on capacity

It is important to note that the degree of hole closure following installation may also affect the holding capacity of dynamically installed anchors (note: anchor capacity is discussed in the next section). An open hole could potentially accelerate the pore water dissipation process thereby reducing the time required from installation to function. However, there would be higher potential for water entrainment along the projectile-soil interface which could reduce frictional resistances over the surface area. In addition, an open hole would imply no bearing capacity at the projectile rear in the conventional API capacity calculations and could hinder the reverse end bearing generated at the tip as proposed by Hossain et al. (2005) for transient suctions beneath spudcans and later for
dynamically installed anchors (Hossain et al. 2014). Furthermore, some anchors (such as the OMNI-Max™) rely on a dive-down/keying mechanism upon extreme loading (Shelton, 2007). An open hole above the anchor may hinder this mechanism and lead to failure in the vertical.

### 2.6 Anchor capacity

As previously outlined, the capacity of a dynamically installed anchor is mainly determined by the soil shear strength in the vicinity of its final embedment depth. This depth is a function of the anchor mass, geometry and impact velocity, and seabed strength profile; with deeper embedments observable in weaker soils and vice versa. For a specified anchor attaining the same impact velocities, comparable soil strengths should therefore be observed in the vicinity of the embedded anchor (and thus comparable capacities), thereby reducing the dependency on strength profiles.

Owing to the geometric similarities of dynamically installed anchors and piles, it is often assumed that anchor capacities can be assessed using conventional techniques for offshore driven piles in soft sediment. Despite the variances in installation rate, the conventional techniques provide a strong basis for considering the expected anchor capacities in the vertical plane.

#### 2.6.1 Ultimate vertical capacity

The American Petroleum Institute (API, 2002) offers a concise and uncomplicated technique for approximating the ultimate vertical tensile capacity \( V_{ult} \) of offshore driven piles in clay. It remains the most prevalent technique employed for estimating dynamic anchor capacity thresholds in the offshore industry and is given by:

\[
V_{ult} = W_s + F_{bear} + F_{frict}
\]

where \( W_s \) is the submerged weight of the pile, \( F_{bear} \) is the total end bearing resistance and \( F_{frict} \) is the total shaft frictional resistance. It should be noted that the API framework specifies that the allowable pile pullout capacity should be limited to \( F_{frict} \). However for ultimate capacity purposes, \( W_s \), end bearing and the soil plug component generating reverse end bearing must be considered.

The resistance forces to vertical loading acting on an embedded dynamically installed anchor are illustrated in Figure 2.32. The end bearing resistance is given by:
where \( N_c \) is a bearing capacity factor (applied to the anchor tip, padeye and top/bottom of the flukes), \( s_u \) is the undrained shear strength at the relevant projected contact area, \( A_p \). \( N_c = 9 \) is generally adopted for pile ends given the geometric similarity to a deeply buried circular footing (Skempton 1951 and API 2002). However for dynamically installed anchors which often feature a pointed tip, \( N_c \) may vary. For instance, \( N_c = 12 \) was adopted by O’Loughlin et al. (2009) and O’Loughlin (2013) for an ellipsoidal tipped DPA. The narrow fluke sections are typically modelled as deeply embedded strip footings using \( N_c = 7.5 \) (Skempton 1951).

The frictional resistance is given by:

\[
F_{frict} = \alpha s_u A_s
\]  

where \( \alpha \) is a dimensionless interface friction ratio (of limiting shear stress to undrained shear strength), \( s_u \) is the undrained shear strength at the relevant surface contact area, \( A_s \).

Figure 2.32 Resistance forces acting on a dynamically installed anchor during vertical loading
The interface friction ratio theoretically ranges over $0 \leq \alpha \leq 1$ and has been determined from empirical correlations with an extensive database of pile test results compiled by Randolph and Murphy (1985). It is expressed as:

$$\alpha = 0.5\Psi^{-0.5} \leq 1 \quad \{\text{for } \Psi \leq 1\}$$

$$\alpha = 0.5\Psi^{-0.25} \leq 1 \quad \{\text{for } \Psi > 1\}$$

where $\Psi$ is defined as:

$$\Psi = \left(\frac{S_{u,ave}}{\sigma'_v}\right)$$

and $\sigma'_v$ is the effective overburden pressure or vertical effective stress.

The short-term fully remoulded friction ratio is often given simply as the inverse of the soil sensitivity; typical kaolin clay sensitivities of 2 - 2.5 giving $\alpha = 0.4$ - 0.5. It is expected that the long term value of $\alpha$ however, following significant post installation consolidation (discussed later), would approach unity. Unfortunately there remains uncertainty regarding the correct value of $\alpha$ to use in capacity calculations for dynamic projectiles. O’Loughlin et al. (2004a) for instance reported good agreement with experimental data using an average $\alpha = 0.8$ while Richardson et al. (2006) found better agreement using an average $\alpha = 0.5$. Both physical modelling studies were performed in normally consolidated kaolin clay samples and allowed for a period of approximately 13 minutes for excess pore pressure dissipation (equivalent to 1 prototype year) following installations. Gilbert et al. (2008) found that by using $\alpha = 1.0$, reasonable predictions against experimental data for piles installed quasi-statically could be made following a relatively long post-installation consolidation time. However, when the piles were dynamically installed (and the same consolidation time implemented) a best fit $\alpha = 0.5$ was found (see Figure 2.33). Furthermore, Richardson et al. (2009) found that $\alpha$ values as high as two were required in theoretical calculations to compare with long-term consolidation tests on model DPAs™.
2.6.2 Inclined loading

Dynamically installed anchors are predominantly being used for taut leg/semi-taut leg mooring systems which offer advantages over other mooring systems in deep to ultra-deep water. A taut mooring leg will typically have angles of between 30 to 50 degrees to the horizontal at the vessel, transmitting both horizontal and vertical load components to the attached embedded anchor. Design of taut-leg moorings is therefore often governed by the vertical holding capacity of the anchoring system as opposed to the lateral capacity for catenary moorings (Ehlers et al. 2004). Consequently, dynamically installed anchor capacity studies have generally been limited to loading in the vertical. Literature regarding their response to inclined loads is scarce and needs to be addressed.

As the padeye is located at the top of most dynamically installed anchors, inclined loading will comprise vertical, horizontal and moment components. The interaction of these components is complex, particularly for the irregular geometry of the anchor.

Medeiros (2002) offers perhaps the only insight for inclined loading of a dynamically installed anchor in the field. 45° loading directly following installation of a 620 kN (dry weight), 12 m long Torpedo Pile in normally consolidated clay was said to provide an increase in ultimate capacity of 2.4 to 2.6 times to that achieved via vertical loading.

Lieng et al. (2000) indicates that DPATM ultimate capacities could increase by approximately 24% and 36% when loaded at 45 degrees to the horizontal, assuming α values of 1 and 0.5 respectively. This is compared to ultimate capacities achievable

Figure 2.33 Comparison of predicted (α = 0.5) versus vertical pull-out capacity for undrained axial loading (after Gilbert et al. 2008)
when loaded exclusively in the vertical. de Sousa et al. (2011) offer similar findings for a numerically modelled Torpedo Pile (see Figure 2.34). In a soil with a prescribed shear strength gradient of 1.5 kPa/m, an increase of approximately 28% was observed (45 degree load vs. 90 degree load) assuming $\alpha = 1$. It should be noted however that both finite element studies used conventional small strain methods and demonstrate ever-increasing load magnitudes, forcing calculations to be stopped at some criterion. Furthermore, reverse end bearing was not considered in either study. Thus the ‘ultimate’ capacities calculated may not offer a strictly accurate representation.

Kay (2013) interpreted inclined capacities for identical criteria as de Sousa et al. (2011) using yield envelope equations. Such equations have been extensively used to capture the soil interaction with plate anchors subjected to inclined loading in soft sediment (e.g. Bransby and O’Neill 1999, Elkhatib and Randolph 2005, Cassidy et al. 2012). General vertical, lateral and moment (VHM) resistance equations previously developed for circular caissons in clay (Kay and Palix, 2010) were modified and found to give very similar capacity results to those reported by de Sousa et al. (2011). This indicates that yield envelope equations offer a useful approach for capacity calculations of dynamically installed anchors subjected to inclined loading.

![Figure 2.34 Load versus normalised displacement of a torpedo anchor; loads in plane 1 and soil A having k = 1.5 kPa/m (after de Sousa et al. 2011)](image)
2.6.3 Consolidation effects

A projectile penetrating soft seabeds leads to considerable disturbance and remoulding of the sediment in its vicinity. Solid driven piles, which are geometrically comparable to dynamically installed anchors, are well known to result in the generation of large excess pore pressures when installed in fine-grained soils (e.g. Randolph 2003). This is due to the combined effects of changes in mean effective stress due to shearing and increases in total stress as soil is forced outwards to accommodate the volume of the pile. These excess pore pressures reduce the effective stresses along a dynamically installed anchor, leading to lower uplift resistances attainable and thus lower anchor capacities in the short term. With the progression over time of consolidation however, there will be a gradual recovery of the soil shear strength accompanied by an increase in effective stress due to dissipation of excess pore pressures in the vicinity of the anchor.

Field measurements of excess pore pressure distributions around driven piles show that the major pore pressure gradients are radial (Bjerrum and Johannessen 1961; Koizumi and Ito 1967; Lo and Stermac 1965; Randolph and Wroth 1979). Kehoe (1989), as reported by Bullock et al. (2005), indicates that the gradual recovery of shear strength of soil in the vicinity of a driven pile is predominantly due to regain in the shaft frictional resistance. Hence, consolidation following dynamic anchor installations is assumed to proceed with the same radial dissipation of excess pore pressures and that the time dependent regain in dynamic anchor capacity is due entirely to the recovery of shaft and fluke frictional resistances.

Consolidation time depends on the volume of soil around a pile from which dissipation must occur. It increases therefore with the (square of the) foundation’s horizontal dimension, D, and decreases with the horizontal coefficient of consolidation, \( c_h \) (Soderberg 1962). The consolidation time, \( t \), is often expressed on a logarithmic scale in terms of a non-dimensional time factor, \( T \), as shown in equation 2.32. This time factor has been incorporated in several dynamically installed anchor capacity studies including Lieng et al. 1999; Richardson et al. 2009; Blake et al. 2015.

\[
T = \frac{c_h t}{D^2} \tag{2.32}
\]

It should be noted that an open-ended/thin-wall pile will displace much less soil than a closed-end/plugged pile. The band of remoulded and heavily stressed soil around a thin-wall pile is greatly reduced, resulting in more rapid consolidation (example field data
for such piles showing significant gains in capacity in the very short term include Jeanjean 2006; Dutt and Ehlers 2009). For such piles, a solid pile of equivalent area may be considered which will generate the equivalent initial field of excess pore water pressure. The equivalent solid pile would therefore have a reduced equivalent diameter, \( D_{eq} \), (Randolph and Gourvenec 2011) of approximately:

\[
D_{eq} = 2\sqrt{D(p_{thick})}
\]

where \( p_{thick} \) is the pile thickness. \( D_{eq} \) would then replace \( D \) in equation 2.32 for thin-wall piles.

Randolph (2003) notes the difficulty and uncertainty in predicting the changes in effective stress during the consolidation process. Calculating the time for the excess pore water dissipation to occur around a pile however is more straightforward. Dissipation curves derived from linear-elastic perfectly plastic cavity expansion theory and consolidation theory are shown in Figure 2.35. The curves can be used to quantify the ratio of current excess pore pressure (\( \Delta u \)) to immediate post-installation excess pore pressure (\( \Delta u_{max} \)). Interestingly, by incorporating the \( D_{eq} \) parameter, it can be seen that all pile types follow the same dissipation trend; the dimensionless period for 90% dissipation (i.e. \( \Delta u / \Delta u_{max} = 0.1 \)), denoted \( T_{90} \), is:

\[
T_{90} = \frac{c_h t_{90}}{D_{eq}^2} \approx 10
\]
Field evidence shows that the capacity of closed ended solid driven piles in clay, with diameters ~0.15 m, can range from 25% to 40% of the ultimate long-term pile capacity promptly following installation (Reese 1955; Karlsrud and Haugen 1983, as cited by Bogard and Matlock 1990, see Figure 2.36). Laboratory tests on flukeless dynamically installed projectiles with lengths (L) ranging from ~15 – 46 cm also indicate short-term vertical capacities of ~30% of the ultimate long-term vertical anchor capacity (Figure 2.37a; Audibert et al. 2006).

Richardson et al. (2009) report centrifuge tests on dynamically installed anchors in soft clay to determine the increase in vertical capacity with time towards ultimate long-term vertical capacity. Zero and four fluke DPA™ pairs were used and results from dynamic installations and ‘equivalent’ quasi-static installations (where anchor release is at the soil surface) were compared. Anchor masses were used such that when the heavier of the two anchors was installed quasi-statically, it achieved similar embedment depths as the lighter anchor installed through dynamic installation. In this way, the effects of the varying installation methodologies on the time-dependent vertical capacity of DPAs™ could be compared objectively. Figure 2.37 shows that the progression of consolidation with time was similar for all the tests, although slightly quicker for the dynamically installed anchors. Capacity predictions from cavity expansion solutions for radial consolidation, based on the analysis of consolidation following the installation of a pile (Randolph and Wroth 1979) are also included on Figure 2.37. The theoretical predictions assume that the initial pore pressure distribution is a function of the rigidity index \( I_r \) of the soil, with typical values of \( I_r \) ranging from 50 to 500 (Randolph 2003). The upper bound value of \( I_r = 500 \) is shown to provide good agreement with the experimental DPAs™ data for dynamic installation; indicating the solution to be a useful method for measuring capacity regain with time for dynamically installed anchors.
Figure 2.36 High short term capacity field evidence for closed end solid driven piles in clay (25% to 40% of the ultimate long-term pile capacity) (after Reese 1955; Karlsrud and Haugen 1983) (as cited by Bogard and Matlock 1990).

Figure 2.37 High short term capacity laboratory evidence for dynamically installed projectiles (after Audibert et al. 2006).
2.7 Summary

By virtue of fast and simple installation through the momentum gained during free-fall, dynamically installed anchors are considered an economically competitive alternative to the conventional deep offshore foundations described in section 2.2. The energy industry is expressing significant interest in the technology, with numerous designs already commercialised and in use for mooring flexible risers, and some MODUs and FPSOs.

However, the chapter has highlighted that there remains a lack of documented experience with the anchors, especially regarding the application of embedment and capacity prediction techniques to experimental data. The primary shortcomings unearthed through the chapter are given below, together with brief descriptions of how they are addressed in the thesis.

- Limited high quality field data indicates a clear need to develop a field experimental database as a means of confirming anchor performance in soft soil environments. To address this shortcoming, an extensive range of tests (greater than 200) on a 1:20 reduced scale dynamically installed anchor was carried out in a lake environment featuring a soft normally consolidated clay bed. The tests encompassed dynamic installation and monotonic loading. Recorded field data were used to develop, calibrate and validate design tools (chapters 6 and 7).

- An analytical theoretical embedment model has been adopted over a range of dynamic studies. The forces that the model must compute iteratively, particularly fluid mechanics drag resistance and strain-rate-dependent shearing resistance, are complex. A major shortcoming in the calibration and validation of the iterative model is that it is generally limited to known starting and end conditions in centrifuge tests rather than continuous motion data over the full dynamic event. To address this, centrifuge installation tests on a model dynamically installed anchor were carried out with continuous motion data captured using a state-of-the-art MEMS accelerometer (chapter 4). A custom made inertial measurement unit (described in chapter 5) allowed for continuous motion data to be captured during the lake environment field installation tests (described in chapter 6).

- The soil response during high speed installations, specifically the hole created in the wake of an advancing projectile remains poorly understood. Whether the
hole will remain open or not was a question first posed in the 1980s when free-falling projectiles were suggested as a means of disposing of nuclear waste. Conflicting evidence from recent dynamic studies gives motivation for carrying out centrifuge tests using a high-speed video camera to capture the impact and early penetration event in soil samples with varying mudline strengths and overconsolidation ratios (chapter 3).

- Finally, a review of important considerations for estimating the capacity of dynamically installed anchors revealed some disparity on calculating vertical uplift capacities with various interface friction ratio values quoted across studies. More importantly, literature regarding the anchor’s response to inclined loading is extremely scarce, highlighting a major shortcoming considering that mooring lines typically arrive at an angle to the anchor padeye. Vertical and inclined monotonic loading tests were carried out in the field to address this shortcoming and the data was used to develop and calibrate capacity prediction tools (chapter 7).
Chapter 3

Soil response in the wake of dynamically installed projectiles

Chapter context: The paper presented in this chapter answers the question as to how the soil responds in the wake of a dynamically installed anchor during dynamic embedment and establishes the conditions under which hole closure does/does not occur (sub-objective 2). This is done through observations and analyses from a centrifuge testing program of model dynamically installed anchors and cylindrical projectiles of varying aspect ratio penetrating soft soil (data from sub-objective 1). Findings from the chapter are incorporated into theoretical embedment predictions in chapters 4 and 6.

This chapter has been published as:

Main objective: Develop calibrated and validated design tools for predicting the performance of dynamically installed anchors in soft soil

Sub-objective 1: Develop an experimental database, encompassing both centrifuge and field tests, for dynamically installed anchors as a means of establishing anchor performance in soft soil and for calibrating design tools

Sub-objective 2: Answer the question as to how the soil responds in the wake of a dynamically installed anchor during dynamic embedment, establishing the conditions under which the hole formed by the passage of the anchor may close

Sub-objective 3: Refine and calibrate analytical embedment models for dynamically installed anchors using motion data collected in the centrifuge and field tests from sub-objective 1

Sub-objective 4: Establish design tools for predicting the monotonic capacity of dynamically installed anchors for a range of load inclinations, calibrated using field data from sub-objective 1

Completion of the above sub-objectives leads to the completion of the main objective and Chapter 8

Chapter 3: Soil response in the wake of dynamically installed projectiles

Chapter 4: Assessing the penetration resistance acting on a dynamically installed anchor in normally and over consolidated clay

Chapter 5: In situ measurement of the dynamic penetration of free-fall projectiles in soft soils using a low-cost inertial measurement unit

Chapter 6: A release-to-rest model for dynamically installed anchors

Chapter 7: Capacity of dynamically installed anchors as assessed through field testing and three dimensional large deformation finite element analyses

Chapter 8: Conclusions derived from the work described in the thesis
Chapter 3

3.1 Abstract

The soil response in the wake of dynamically installed (free-fall) projectiles is poorly understood, notably with respect to the potential for the hole the projectile creates during dynamic penetration to remain open. This paper considers this problem through centrifuge tests in which impact of free-falling projectiles on the surface of kaolin clay was captured using a high speed video camera. The video observations show that hole closure may occur at the same rate as the projectile penetrates, or may remain open, either fully or partially. The paper shows that hole closure is controlled by a dimensionless strength ratio, expressed in terms of the undrained shear strength at the rear of the embedded projectile, the projectile diameter and the effective unit weight of the soil. The centrifuge data agree well with an expression derived from tests on undrained, constant rate of penetration spherical penetrometer tests, demonstrating that hole closure is controlled by soil backflow at the rear of the projectile, regardless of the geometrical aspect ratio (length/diameter) of the projectile. This expression can then be used to assess hole closure assumptions made in dynamic penetration analyses, by comparing the final embedment depth with the calculated transitional depth for soil backflow.
3.2 Introduction

Projectiles that are dynamically installed in a water environment include penetrometers for measuring soil strength and anchors for mooring floating facilities. Challenges associated with understanding the dynamic installation process include quantifying the enhancement of soil strength over the extreme range of strain rates evoked in the soil and accounting for hydrodynamic effects that dominate during shallow penetration. These issues have been addressed experimentally and numerically (e.g. O’Loughlin et al. 2004, 2009, 2013, 2014; Einav et al. 2004; Aubeny and Shi, 2006; Raie and Tassoulas, 2006; Nazem et al. 2012; Gaudin et al. 2013; Chow et al. 2014; Steiner et al. 2014; Blake and O’Loughlin, 2015).

A further complication relates to soil behaviour in the projectile wake during dynamic installation and the potential for the hole it creates to remain open. This uncertainty has an influence on the net penetration resistance during installation, as an open hole will increase soil buoyancy and decrease bearing resistance due to the absence of reverse end bearing at the trailing end of the projectile. Although these are compensating effects, the relative magnitude of buoyancy compared to reverse end bearing is specific to the projectile geometry. Quantifying the net penetration resistance is crucial for strength measurement projectiles, as the undrained shear strength is calculated directly from the net penetration resistance, and also for dynamically installed anchors as the eventual holding capacity is controlled by the final embedment depth, which is calculated from the net penetration resistance.

There are also consequences for post installation behaviour. An open hole above a dynamically installed anchor will reduce the contribution from bearing resistance to anchor capacity, whilst potentially accelerating dissipation of pore pressures that develop during dynamic installation if a boundary water layer exists at the projectile-soil interface. For anchors that are designed to dive into deeper (and stronger) soil when loaded (e.g. Shelton 2007), there is higher potential for the anchor to move vertically upwards if an opening exists above the anchor, particularly in stronger soils in which the inclination of the mooring line is likely to be relatively steep.

Some experimental studies have assumed that the hole created by the passage of a dynamically penetrating projectile would remain open, at least over the duration taken for installation (Aubeny and Shi, 2006; O’Loughlin et al., 2013; Chow et al, 2014). However, other studies assumed that the hole closes either during or shortly after
installation (e.g. Burdett and Karnes 1986; Pooroshaab and James 1989). The disparity is supported by finite element studies reported by Sabetamal et al. (2013) that showed immediate hole closure (i.e. during dynamic installation) for a normally consolidated soil, but open holes for a soil with an overconsolidation ratio of 2.

The hole closure question is considered in this paper through centrifuge tests in which the soil response in the wake of dynamically installed projectiles is captured using a high speed camera. The contribution of the paper is experimental evidence that for the first time shows that hole closure may occur at the same rate as the projectile penetrates, but that the process is governed by the dimensionless soil strength ratio at the rear of the projectile, expressed in terms of the undrained shear strength, the hole diameter and the effective unit weight of the soil.

3.3 Experimental details

The problem was addressed through centrifuge tests in which projectiles were dynamically installed in kaolin clay with the impact and initial penetration event captured using a high speed camera (MotionBLITZ Cube 2) at a frame rate of 2.27 kHz (see Figure 3.1). The clay was prepared by consolidating kaolin slurry under self-weight in the centrifuge at an acceleration of either 100 g or 200 g and the tests carried out at the same acceleration used during consolidation. Variations in the strength profile of the samples were achieved by periodically scraping a predetermined depth of clay from the sample surface, which increased the strength of the clay at the mudline and the overconsolidation ratio.

The projectiles included 6 mm diameter brass cylindrical models with aspects ratios L/D (length/diameter) in the range 1 to 6 and hemispherical tips and rears (such that the model with L/D = 1 is a sphere, see Figure 3.2a), and a 75 mm long, 6 mm shaft diameter (L/D = 12.5) aluminium-brass composite dynamically installed anchor (see Figure 3.2b). The projectiles were dynamically installed by allowing them to fall from model heights of 36 mm to 50 mm (cylindrical models) and approximately 300 mm (model anchors) through a vertical installation guide above the clay surface. A layer of free water measuring 55 mm to 100 mm (cylindrical model) and approximately 30 mm (model anchor) was maintained during the tests. Resulting impact velocities, as assessed from photo-emitter-receiver pairs (PERPs, Richardson et al., 2006) located along the installation guide, and images captured using the high speed camera, were
approximately 10 m/s for the cylindrical projectiles and 16 m/s for the dynamically installed anchor (see Table 3.1).

3.4 Results and discussion

3.4.1 Undrained shear strength

Typical $s_u$ profiles determined from T-bar penetrometer tests are shown in Figure 3.3, where the depth axis reflects the changing mudline surface with each scrape, represented by the sum of the current sample depth, $z$, plus the scrape depth, $\Delta z_{\text{scrape}}$. The $s_u$ profiles were interpreted from the measured penetration resistance according to the method proposed by White et al. (2010), which accounts for soil buoyancy and shallow embedment effects that extend to deeper depths in overconsolidated soils.

The strength of the samples can be assumed to vary with depth according to the relationship proposed by Ladd et al. (1977):

$$s_u = \sigma'_v \left( \frac{s_u}{\sigma'_v} \right)_{\text{nc}} \text{OCR}^\Lambda$$

where $\sigma'_v$ is the current vertical effective stress determined from the profile of effective unit weight, $\gamma'$, with depth established from post-testing sample cores and the (slightly) varying acceleration level with sample depth, and $\Lambda$ is the plastic volumetric strain ratio (Schofield and Wroth, 1968). The best fit between Equation 3.1 and the normally consolidated $s_u$ profile on Figure 3.3 was obtained using $(s_u/\sigma'_v)_{\text{nc}} = 0.15$, which is typical for centrifuge kaolin samples (Chow et al., 2014; Morton et al., 2014; Hu et al., 2014) and corresponds to an undrained shear strength gradient, $k = 1$ kPa/m. The overconsolidated samples were best fitted using $\Lambda = 0.9$ in Equation 3.1, which is at the upper end of the typical range, $\Lambda = 0.7$ to 0.9 (Mitchell and Soga, 2005). Undrained shear strength ratios for the 200 g normally consolidated samples were higher, in the range $(s_u/\sigma'_v)_{\text{nc}} = 0.22$ to 0.27 ($k = 1.5$ to 1.9 kPa/m, see Table 3.1).

3.4.2 Evidence of both open and closed holes

For objects that are penetrated at nominally undrained rates, hole closure is a function of the dimensionless strength ratio, $s_u/\gamma'D$, where $s_u$ is the undrained shear strength, generally defined at the depth at which the hole closes and $D$ is the hole diameter. This has been convincingly demonstrated for spudcans (Hossain et al., 2005), pipelines and T-bar penetrometers (White et al., 2010; Tho et al., 2012) and ball penetrometers (Zhou...
et al., 2013; Morton et al., 2014). A common theme across these studies is the mechanism that governs hole closure, which is now understood to be soil backflow at the rear of the penetrating object and not wall collapse of the hole formed during installation.

During initial penetration, soil displaced by the advancing projectile will be projected outwards and upwards causing heave at the mudline. As penetration progresses the mechanism is controlled by a combination of spherical and cylindrical cavity expansion (Randolph, 2003), depending on the projectile geometry. Once the rear of the projectile passes below the mudline, an open cavity may form or backflow may occur depending on the local $s_u/\gamma'D$. However, if a cavity does form, it is expected that this will only remain to a critical penetration depth, again controlled by $s_u/\gamma'D$, beyond which soil will backflow behind the advancing projectile.

Visual evidence from images taken of the impact site after dynamic installation, but whilst the centrifuge was still spinning, are presented in Figure 3.4. The installations relate to tests with the model anchor shown in Figure 3.2b, which achieved an average impact velocity, $v_i = 16$ m/s. Three conditions were noted following installation. The first (Figure 3.4a) was full hole closure with a slight surface depression, observed in tests with the lowest soil strengths behind the projectile ($s_{um} = 0$ and 1.2 kPa; $s_u = 2.6$ to 3.7 kPa at $z_e$, where $z_e$ is the embedment depth measured to the rear of the projectile). The second (Figure 3.4b) was partial hole closure, evident as open holes with a diameter less than the projectile diameter and with a depth that did not extend to the rear of the projectile. Partial hole closure was observed in tests with $s_{um} = 1.9$ to 2.5 kPa and $s_u = 4.1$ to 4.5 kPa at $z_e$. The third condition (Figure 3.4c) was an open hole with a diameter equal to that of the projectile, observed in tests with the highest strengths behind the projectile ($s_{um} = 2.8$ to 4.1 kPa; $s_u = 4.3$ to 5.7 kPa at $z_e$). From this visual evidence it appears that the potential for soil backflow is controlled by the soil strength at the rear of the projectile, or an average soil strength between the rear of the projectile and the mudline. As the projectile passes below the mudline, the increased lateral stresses caused by the cavity expansion reduce, allowing the soil to move inwards into the cavity. This is most evident in the high speed video that accompanies this paper, where in the case of a fully closed hole, the surface heave caused during initial penetration translates into the void created by the advancing projectile. However, finite element studies reported by Sabetamal et al. (2014) show that transient suction forces exist in the wake of the dynamically penetrating projectiles, which may also ‘pull’ soil into the
cavity. These suction forces may be sufficient to cause the reduction in the hole diameter in the tests where partial hole closure was observed, but not in the tests where a fully open hole was observed.

### 3.4.3 Quantifying the depth at which hole closure occurs

The final embedment depths at the rear of the projectile, $z_e$, and the corresponding values of $s_u/\gamma'D$ (at $z_e$) are shown in Figure 3.5. The data are grouped according to the three observed conditions at the impact site (fully closed hole, partially closed hole and open hole), which makes it clear that the depth to which a hole will remain open increases as $s_u/\gamma'D$ increases. Also shown on Figure 3.5 is the expression proposed by Morton et al. (2014) for the dimensionless transitional depth, $z_{deep}/D$, at which a full-flow failure mechanism will develop for a ball penetrometer:

\[
\frac{z_{deep}}{D} = a + b \left( \frac{s_u}{\gamma'D} \right)^c + \frac{d - a}{1 + [(s_u/\gamma'D)/e]^f}
\]  

(3.2)

where the fitting constants $a = 16.3$, $b = 0.12$, $c = 1.3$, $d = 0.52$, $e = 4.9$ and $f = 1.5$.

Equation 3.2, which was established from undrained, constant rate of penetration tests using a ball penetrometer, is seen to acceptably predict the embedment depth at which hole closure will occur for a dynamically installed projectile. The embedment depths on Figure 3.5 do not represent the depth at which backflow occurs (as this is not measurable in the dynamic penetration tests) but rather the final depths at the rear of the projectile when it comes to rest in the soil. Hence, data points above the line indicate that soil backflow occurred before the projectile came to rest, data points on the line indicate that the projectile has come to rest at the embedment depth where soil backflow is about to occur, and data points below the line indicate that the final embedment depth is insufficient for soil backflow. It is worth noting that the tests in which partial hole closure was observed are expected to lie above the line, as these tests are also characterised by soil backflow, albeit to a limited height above the rear of the projectile. Figure 3.5 also shows that the hole closure response is independent of aspect ratio. This is to be expected as the mechanism attributed to hole closure is backflow at the rear of the projectile, which should be similar for a low or high aspect ratio.

### 3.4.4 Rate of hole closure

Another aspect of the problem relates to the instances where hole closure does occur, and whether this will be at the same rate as the projectile penetrates. Figure 3.6 shows
time progression of a projectile from the point of impact at the mudline until the point where the rear of the projectile is considered to have just passed the mudline. The images were acquired at 2.27 kHz, such that the time between successive images is ~0.44 ms. Figure 3.6, which relates to Test 5 using a cylindrical projectile with L/D = 6, indicates that the rear of the projectile passed the mudline in image 3 (t = 0 ms) and that full hole closure was complete by image 6, ~1.76 ms later. This is much less than the time required for the projectile to come to rest, which cannot be less than 5.8 ms (based on an impact velocity of 9.1 m/s and a travel distance of 53 mm). Hence, it is reasonable to conclude that where hole closure will occur, it may take place during dynamic penetration. In such instances the net penetration resistance should consider reverse end-bearing at the rear of the projectile and limit the soil buoyancy to that calculated using the volume of the projectile (i.e. not considering additional volume from an open hole above the projectile).

### 3.5 Conclusions

This paper provides evidence that the soil in the wake of a dynamically installed projectile may close, either fully or partially, or remain open depending on the dimensionless strength ratio, $s_u/\gamma'D$ at the rear of the at-rest projectile. In instances where hole closure was observed, high speed video capture of the impact site showed that the hole closes whilst the projectile is penetrating and not after it comes to rest.

The transitional depth for soil backflow and hence the onset of hole closure may be calculated for a dynamic penetration event using an expression developed for interpreting ‘push-in’ ball penetrometer tests. This expression can then be used to assess hole closure assumptions made in dynamic penetration analyses, by comparing the final embedment depth with the calculated transitional depth for soil backflow.
### 3.6 Tables

Table 3.1 Summary of test results: (a) cylindrical projectiles (200 g) (b) dynamically installed anchors (100 g)

(a)

<table>
<thead>
<tr>
<th>Description</th>
<th>Test</th>
<th>Sample</th>
<th>Aspect ratio, L/D</th>
<th>Prototype drop height above malline, z (m)</th>
<th>Impact velocity, v (m/s)</th>
<th>Prototype embedment depth at rear of projectile, z_s (m)</th>
<th>Undrained shear strength at midline, s_u (kPa)</th>
<th>gradient, k (kPa/m) at z_s, s (kPa)</th>
<th>Effective unit weight at z_s, γ' (kN/m^3)</th>
<th>Hole</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cylindrical projectiles of varying aspect ratio, installed at 200 g</td>
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<td>1</td>
<td>10.4</td>
<td>8.62</td>
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<td>1.5</td>
<td>7.8</td>
<td>6.6</td>
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<td></td>
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(b)

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<th>Description</th>
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<th>Impact velocity, v (m/s)</th>
<th>Prototype embedment depth at rear of projectile, z_s (m)</th>
<th>Undrained shear strength at midline, s_u (kPa)</th>
<th>gradient, k (kPa/m) at z_s, s (kPa)</th>
<th>Effective unit weight at z_s, γ' (kN/m^3)</th>
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3.7 Figures

(a)
Figure 3.1 Centrifuge testing arrangements: (a) schematics for (1) cylindrical projectile tests at 200 g, (2) model anchor tests at 100 g, and (b) cylindrical projectile testing arrangement.
Figure 3.2 Model projectiles: (a) cylindrical projectiles with varying aspect ratios and hemispherical tips and rears, (b) dynamically installed anchor featuring an elliptical tip and four large flukes (modelled on an equivalent prototype anchor; Lieng et al. 1999).
Figure 3.3 Undrained shear strength profiles for soil samples consolidated (100 g).
Figure 3.4 Projectile impact sites after penetration of dynamically installed anchors: (a) full hole closure, (b) partial hole closure, and (c) open hole.
Figure 3.5 Dependence of final projectile embedment depth (rear of projectile) on dimensionless strength ratio, $s_u/\gamma'D$. Reference numbers 1, 1.5, 2, 4 and 6 adjacent to the projectile data points for tests 1 to 11 indicate the aspect ratio. Colour codes: blue = full hole closure, red = partial hole closure, green = fully open hole.
Figure 3.6 High speed video capture of a projectile impacting clay (Test 5, L/D = 6, $s_{um} = 0.5$ kPa)

Refer also to online video:
Chapter 4

Assessing the penetration resistance acting on a dynamically installed anchor in normally and over consolidated clay

Chapter context: The paper presented in this chapter serves to calibrate and reinforce the merit of an analytical model used to predict the motion response of dynamically installed anchors penetrating soil (addressing sub-objective 3). Centrifuge test data from sub-objective 1 using a model dynamically installed anchor fitted with an internal micro-electro mechanical system (MEMS) are analysed and used to validate the model. Coupled with the soil response findings from chapter 3, this chapter aids the development of Chapter 6 via complete experimental motion data profiles obtained in a controlled environment.

This chapter has been accepted for publication as:

Main objective: Develop calibrated and validated design tools for predicting the performance of dynamically installed anchors in soft soil.

Sub-objective 1: Develop an experimental database, encompassing both centrifuge and field tests, for dynamically installed anchors as a means of establishing anchor performance in soft soil and for calibrating design tools.

Chapter 3: Soil response in the wake of dynamically installed projectiles.

Sub-objective 2: Answer the question as to how the soil responds in the wake of a dynamically installed anchor during dynamic embedment, establishing the conditions under which the hole formed by the passage of the anchor may close.

Chapter 4: Assessing the penetration resistance acting on a dynamically installed anchor in normally and over consolidated clay.

Sub-objective 3: Refine and calibrate analytical embedment models for dynamically installed anchors using motion data collected in the centrifuge and field tests from sub-objective 1.

Chapter 5: In situ measurement of the dynamic penetration of free-fall projectiles in soft soils using a low-cost inertial measurement unit.

Sub-objective 4: Establish design tools for predicting the monotonic capacity of dynamically installed anchors for a range of load inclinations, calibrated using field data from sub-objective 1.

Chapter 6: A release-to-rest model for dynamically installed anchors.

Completion of the above sub-objectives leads to the completion of the main objective and Chapter 8.

Chapter 7: Capacity of dynamically installed anchors as assessed through field testing and three dimensional large deformation finite element analyses.

Chapter 8: Conclusions derived from the work described in the thesis.
4.1 Abstract

Predicting the final embedment depth of a dynamically installed anchor is a key prerequisite for reliable calculation of anchor capacity. This paper investigates the embedment characteristics of dynamically installed anchors in normally and over consolidated clay through a series of centrifuge tests involving a model anchor instrumented with a MEMS accelerometer, enabling the full motion response of the anchor to be established. The data are used to assess the performance of an anchor embedment model based on strain-rate-dependent shearing resistance and fluid mechanics drag resistance. Predictions of a database of over 100 anchor installations – formed from this study and the literature – result in calculated anchor embedment depths that are within ±15% of the measurements. An interesting aspect, consistent across the entire database, relates to the strain rate dependence on frictional resistance relative to bearing resistance. The predictions reveal that strain rate dependency may indeed be higher for frictional resistance, although only if a soil strength lower than the fully remoulded strength is considered as the reference strength, which suggests that water may be entrained along a boundary layer at the anchor-soil interface during installation.
4.2 Introduction

The shift of oil and gas operations towards more remote locations, often in deeper water, tends to increase the costs associated with installing the anchors and laying the moorings. As much of this cost is due to the day-rate of installation vessels, there is an incentive to reduce the overall anchor installation durations, particularly for mobile drilling operations. This may be achieved by using anchors that are quick to install, such as dynamically installed anchors that free-fall through the water column and self-bury into the seabed (Figure 4.1). Various geometries have been proposed, although they tend to be rocket or torpedo shaped with overall lengths between 9 and 22 m, dry weights in the range 25 to 120 t and featuring three or four flukes (also referred to as fins) at the trailing edge.

Anchor capacity is derived from a combination of frictional resistance along the anchor length, bearing resistance on the anchor flukes and the submerged weight of the anchor. Evidently the final anchor embedment depth becomes a key design parameter as this will dictate the soil strength in the vicinity of the embedded anchor and in turn the anchor capacity. Predicting this final embedment depth is challenging due to the very high penetration velocities (up to ~30 m/s) that: (i) significantly increase the mobilised soil strength during dynamic embedment, and (ii) introduce hydrodynamic aspects, including potential water entrainment at the anchor-soil interface and pressure drag that occurs both during free-fall in water and during dynamic embedment in soil (O’Loughlin et al., 2014a).

Theoretical approaches for predicting the final anchor embedment depth have been proposed and their merit has been examined using data, mainly from centrifuge studies (e.g. O’Loughlin et al. 2004a, 2009, 2013; Richardson et al. 2006; Gaudin et al. 2013) in normally consolidated clay. However, limitations on the measurements available during dynamic penetration have limited the calibration and validation of these embedment models to known starting and end conditions: the measured anchor impact velocity (at zero embedment) and the measured anchor embedment (at zero anchor velocity). Although this has been an important ‘first step’ in demonstrating the appropriateness of such models in predicting final embedment, an equally important ‘second step’ is to rigorously appraise these models using motion data measured during the dynamic embedment event. The complexity of the mechanisms that occur during dynamic embedment warrants this development, offering a basis for assessing parameter selection and the inclusion or exclusion of the various resistance terms. This
paper adopts such an approach, extending previous work by considering the motion response during penetration, but also by considering both normally and overconsolidated clay.

4.3 Experimental details

The motion data for further validating the anchor embedment model was obtained in centrifuge tests using a dynamically installed anchor instrumented with a micro-electro mechanical system (MEMS) accelerometer. The tests were conducted in both normally and overconsolidated clay so that the model could be applied to a range of overconsolidation ratios and strength profiles. Relevant experimental details are provided in the following sections.

4.3.1 Soil samples

Clay samples were prepared by consolidating kaolin slurry, with an initial moisture content of twice the liquid limit (120%), under self-weight in a beam centrifuge at an acceleration of 100 g for six days. During this time additional slurry was added to the sample surfaces to maintain a target sample height of ~ 220 mm. A 30 mm water layer was maintained at the sample surface to ensure sample saturation during consolidation and testing. The tests were conducted in three sample boxes, each with internal dimensions 598 mm long, 117 mm wide and 300 mm deep, nested side by side in a larger strongbox. A geotextile drainage blanket was placed along the base of the sample boxes and along the end walls to ensure two-way drainage during consolidation and that there was no hydraulic gradient between the top and base of the samples. Variations in the strength profile of the clay samples were achieved by periodically scraping a predetermined depth of clay from the sample surface and allowing a swelling period of approximately 15 hours before further testing. This had the effect of increasing both the strength of the clay at the mudline and the overconsolidation ratio (OCR).

4.3.2 Instrumented model anchor

The centrifuge tests were carried out at the consolidation acceleration level of 100 g using the model anchor shown in Figure 4.2. The model anchor was fabricated from aluminium and brass, and was based on an idealised geometry proposed by Lieng et al. (1999), with an elliptical tip and four large flukes at the trailing end. At model scale the anchor has a length, L = 75 mm, a shaft diameter, D = 6 mm, and a mass, m = 10.17 g.
At prototype scale the anchor length and mass is 7.5 m and 10.17 tonnes respectively, representing a half scale full size anchor. The model scaling was based on constraints on the maximum size that could be accommodated in the centrifuge and the maximum measurement range of the accelerometer used to measure the motion response during dynamic embedment.

The model anchor was instrumented with a single axis ± 500 g MEMS accelerometer (Analog Devices ADXL001) measuring 5 × 5 × 2mm, that was embedded within an epoxy resin filled void in the anchor shaft (as shown in Figure 4.2). The MEMS accelerometer can measure both constant and changing acceleration and was used to capture the motion response of the anchor during free-fall and embedment in soil. The three sensor wires (power, ground and signal) were recessed into the anchor shaft and exited at the rear of the shaft where they joined the anchor retrieval line.

4.3.3 Experimental arrangement and procedures

The experimental arrangement is shown in Figure 4.3; as it is similar to that described in detail by O’Loughlin et al. (2004a), only a brief description is provided here. The anchor was dynamically installed by allowing it to fall from a preselected height above the sample surface (typically 300 mm) through a vertical installation guide (which applied the tangential force required to keep the anchor rotating at the same angular velocity as the centrifuge). Anchor release was achieved in-flight by supplying current to a resistor that caused it to heat and burn through a sacrificial release cord. Anchor velocity was primarily derived from the MEMS accelerometer measurements, although independent checks on the accelerometer-derived anchor velocity before impact with the soil was achieved using photo-emitter-receiver pairs (PERPs; see Richardson et al. 2006, Chow et al. 2014) that were located on the lower end of the installation guide (i.e. close to the sample surface). The PERPs also served as a pre-trigger for the accelerometer data that was captured at 50 kHz using the high-speed logging mode in the data acquisition hardware (Gaudin et al. 2009) when the anchor passed the first PERP. After dynamic embedment the centrifuge was spun down and a direct measurement of anchor embedment was taken by clamping the retrieval line from the rear of the anchor at the sample surface, extracting the anchor, and then measuring the vertical distance from the clamp to the anchor tip using a scale rule marked in half millimetre divisions.
4.4 Results and discussion

4.4.1 Undrained shear strength

Profiles of undrained shear strength, \( s_u \), were determined for each sample from T-bar penetrometer tests, involving a penetration velocity, \( v = 1 \text{ mm/s} \), such that over the penetration depth the non-dimensional velocity \( v_d/c_h = 10 \) to 50 (\( d \) is the T-bar diameter = 5 mm and \( c_h \) is the coefficient of horizontal consolidation \( \approx 0.1 \) to 0.5 \( \text{mm}^2/\text{s} \)) (Colreavy et al. 2016) and the response is primarily undrained (House et al., 2001). Typical \( s_u \) profiles are shown in Figure 4.4, where the depth axis reflects the changing mudline surface with each scrape, represented by the sum of the current sample depth, \( z \), plus the scrape depth, \( \Delta z_{\text{scrape}} \). Also shown on Figure 4.4 are profiles of OCR with depth, which (with the exception of the normally consolidated sample, for which OCR = 1) reduce non-linearly with depth from OCR = 3.5 to 10.2 at 6 mm below the mudline (one anchor shaft diameter, D) to OCR = 1.1 to 1.6 at \( z = 150 \text{ mm} \). The profiles were interpreted from the measured penetration resistance according to the framework proposed by White et al. (2010), which accounts for soil buoyancy and a capacity factor that evolves from \( N_c = 4.5 \) at shallow embedment to \( N_c = 10.5 \) at an embedment depth when a full-flow mechanism develops. This transitional embedment depth increases with the dimensionless strength, \( s_u/\gamma'D \), and consequently the shallow embedment interpretation becomes more important in over consolidated soils.

Also shown on Figure 4.4 are \( s_u \) profiles determined using the classical expression for overconsolidated soil strength (Ladd et al. 1977):

\[
s_u = \sigma'_v \left( \frac{s_u}{\sigma'_v} \right)_{nc} \text{OCR}^\Lambda
\]

where \( \sigma'_v \) is the current vertical effective stress determined from the profile of effective unit weight, \( \gamma'_v \), with depth established from post-testing sample cores and the (slightly) varying acceleration level with sample depth, and \( \Lambda \) is the plastic volumetric strain ratio (Schofield and Wroth, 1968). The best fit between Equation 4.1 and the experimental \( s_u \) profiles on Figure 4.4 was obtained using \( (s_u/\sigma'_v)_{nc} = 0.14 \) to 0.18 (increasing slightly with centrifuge spinning time) which is typical for centrifuge kaolin samples (Chow et al., 2014; Morton et al., 2014; Hu et al., 2014) and \( \Lambda = 0.85 \). The latter is at the upper end of the typical range, \( \Lambda = 0.7 \) to 0.9 (Mitchell and Soga, 2005), and is perhaps reflective of the limited swelling period between scraping the soil surface and the subsequent T-bar test. The theoretical \( s_u \) profiles may alternatively be expressed as \( s_u = \)
s_{um} + kz, where the mudline strength intercept, s_{um} = 0 to 4 kPa and k = 0.9 to 1.2 kPa/m (depending on OCR).

4.4.2 Interpretation of accelerometer data

The linear acceleration, $a_{linear}$, due to the motion of the anchor may be deduced from the accelerometer measurements (that also include a component of centripetal acceleration) following the methodology outlined by O’Loughlin et al. (2014b). Figure 4.5 plots profiles of anchor velocity, $v$, and linear acceleration, $a_{linear}$, with displacement, $z$, where $v$ and $z$ were calculated by numerically integrating $a_{linear}$ (once for velocity and twice for displacement). As recommended by O’Loughlin et al. (2014b), the mudline was established by matching the measured anchor embedment depth and the anchor displacement calculated from the numerical integration. The total anchor displacement was calculated as 416 and 415 mm, which corresponds to free-fall distances of 306 mm (Test 2) and 330 mm (Test 15) compared with the respective preselected drop heights of 308 mm and 332 mm. These small variations were anticipated and are due to extension of the sacrificial release cord as the centrifuge spins up to 100 g. Confidence in the motion data derived from the MEMS accelerometer measurements may also be drawn from the good agreement with the independent velocity measurements obtained from the PERPs.

Figure 4.5 shows that the theoretical linear acceleration of the anchor (based on the anchor acceleration relative to the sample surface) ignoring friction that develops at the anchor-guide interfaces overestimates the anchor linear acceleration and velocity by up to 35 and 38 % respectively. The actual linear acceleration of the anchor can be estimated by discounting the ‘zero-friction’ linear acceleration, $r_0^2$, by an amount due to the friction along the guide, $2\mu rv$, where $\mu$ is the dynamic coefficient of friction (taken as 0.45 for brass/aluminium sliding on steel) and $v$ is the anchor velocity (Chikatamarla et al. 2006).

The resulting acceleration profile obtained using this adjustment is also shown on Figure 4.5. The agreement with the experimental data is good, with the exception of approximately 40 mm after release where additional resistance (perhaps from the sequential parting of the strands of the release cord, which hindered instantaneous release) causes the linear acceleration derived from the measurements to be lower. Nevertheless, the theoretical linear acceleration satisfactorily quantifies the frictional
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resistance due to the guide that must be considered over the first anchor length of soil penetration as discussed later in the paper.

4.5 Anchor embedment model

Data such as those presented on Figure 4.5 are important for the validation of models for predicting the embedment of dynamically installed anchors. This study considers an anchor embedment model first proposed by O’Loughlin et al. (2004b). Details of this model are provided, before examining its merit by comparing predictions with the centrifuge data considered here.

4.5.1 Model formulation

The model considers the forces acting on the anchor in one-dimensional space (depth) and time and equates the net force on the anchor, \( F_{\text{net}} \), to the product of the anchor mass and acceleration. This is essentially Newton’s second law of motion, which for the anchor problem considered here may be expressed as:

\[
F_{\text{net}} = \left( m + m' \right) \frac{d^2z}{dt^2} = W - F_b - R_f \left( F_{\text{frict}} + F_{\text{bear}} \right) - F_d
\]

where \( m \) is the anchor mass, \( m' \) is the added mass of soil that is accelerated with the anchor, \( z \) is anchor depth, \( t \) is time, \( W \) is the anchor dry weight, and the resistance forces acting on the anchor (shown in Figure 4.6) are: the buoyancy force, \( F_b \), equal to the weight of the soil displaced by the anchor, frictional resistance along the soil anchor interface, \( F_{\text{frict}} \), bearing resistance on the anchor tip and flukes, \( F_{\text{bear}} \), and drag resistance, \( F_d \). The added mass, \( m' \) in Equation 4.2 is an important consideration for geometries with low aspect ratios (e.g. spheres, Morton et al. 2015) but for long slender bodies such as the anchor considered here, \( m' \) is negligible and can be taken as zero (Beard 1981, Shelton et al. 2011).

As will be described shortly, the \( R_f \) term in Equation 4.2 accounts for the viscous enhancement of strength due to strain rate effects, but does not describe the pressure drag component of drag resistance, which is independent of viscous effects and is linked to the stagnation pressure (Zhu and Randolph, 2011; Randolph and White 2012; Blake and O’Loughlin, 2015). This drag resistance is well known in fluid mechanics but has also been shown to be an important – and in some instances a dominating – component
of resistance for geotechnical problems (Zhu and Randolph, 2011; Randolph and White, 2012; Sahdi et al. 2014a; Morton et al. 2015). Drag resistance is formulated as:

\[ F_d = \frac{1}{2} C_D \rho_s A_p v^2 \]  \hspace{1cm} (4.3)

where \( C_D \) is a drag coefficient, \( \rho_s \) is the soil density, \( A_p \) is the anchor’s projected area and \( v \) is the anchor velocity.

Frictional and bearing resistances can be expressed in the form:

\[ F_{frict} = \alpha s_u A_s; \quad F_{bear} = N_c s_u A_p \]  \hspace{1cm} (4.4)

where \( \alpha \) is an interface friction ratio (of limiting shear stress to undrained shear strength), \( N_c \) is the bearing capacity factor for the anchor tip or fluke and \( s_u \) is the undrained shear strength averaged over the appropriate contact area – frictional area, \( A_s \), or projected area, \( A_p \).

As mentioned previously, the high anchor penetration velocity leads to strain rates in the soil that are beyond that associated with undrained behaviour and increase soil strength (Casagrande & Wilson 1951, Graham et al. 1983, Sheahan et al. 1996). As shown by Equation 4.2, these viscous strain rate effects are accounted for by employing a strain rate function, \( R_f \), to scale the undrained shear strength according to the shear strain rate. \( R_f \) is generally described using either power or semi-logarithmic functions. The power law is generally preferred for problems involving higher orders of magnitude variation in strain rate (Biscontin and Pestana, 2001; Abelev and Valent 2009; O’Loughlin et al. 2013) and may be formulated as:

\[ R_f = \left( \frac{\dot{\gamma}}{\dot{\gamma}_{ref}} \right)^\beta \]  \hspace{1cm} (4.5)

where \( \dot{\gamma} \) is the strain rate, \( \dot{\gamma}_{ref} \) is a reference strain rate associated with the measurement of undrained strength and \( \beta \) is a strain rate parameter. Although quantifying the absolute magnitude of strain rate and its variation within the shear zone is non-trivial, it is reasonable to assume that at any given location, the operational strain rate is proportional to \( v/D \), such that:

\[ R_f = \left( \frac{v}{D} \right)^\beta \]  \hspace{1cm} (4.6)
where \((v/d)_{\text{ref}}\) in this instance is calculated using the T-bar penetration velocity, \(v\), and diameter, \(d\).

A further aspect of Equation 4.2 relates to the question of hole closure in the wake of the anchor during dynamic penetration. As discussed by O’Beirne et al. (2015a) and shown by Figure 4.6, if the hole formed by the passage of the anchor closes immediately after penetration, additional reverse end-bearing resistance should be included at the rear of the anchor fluke and shaft and the soil buoyancy, \(F_b\), should be limited to that calculated using the volume of the anchor (i.e. not considering additional volume from an open shaft hole above; Figure 4.6, option 2). Conversely, an open hole would require soil buoyancy to be calculated using a volume equal to that of the anchor plus a cylinder with a height equal to the distance from the rear of the anchor to the mudline (assuming the slots formed by the flukes will always close owing to their low thickness, O’Loughlin et al. (2013)) and not include additional reverse end bearing resistance at the rear of the anchor (Figure 4.6, option 1).

As shown by Morton et al. (2014) for pushed-in penetrometers and O’Beirne et al. (2015a) for dynamically penetrating projectiles, the cylindrical hole formed by the passage of a cylindrical or spherical body is expected to close at a transitional depth, \(z_{\text{deep}}\), controlled by the dimensionless strength ratio, \(s_u/\gamma'D\), where \(s_u\) and \(\gamma'\) are the local undrained shear strength and effective unit weight at the rear of the body. This ‘hole-closure depth’, \(z_{\text{deep}}/D\), may be quantified from

\[
\frac{z_{\text{deep}}}{D} = a + \left(\frac{b s_u}{\gamma' D}\right)^c + \frac{d - a}{1 + [(s_u/\gamma'D)/e]^f}
\]

where the fitting constants \(a = 16.3\), \(b = 0.12\), \(c = 1.3\), \(d = 0.52\), \(e = 4.9\) and \(f = 1.5\).

Equation 4.7 may be used in an embedment prediction by firstly making an assumption regarding hole closure and modelling buoyancy and bearing resistance accordingly. The calculated final embedment depth at the rear of the anchor would then be used to calculate \(z_{\text{deep}}/D\) and in turn, to check the validity of the initial assumption, allowing a revised calculation (using the opposing hole closure condition) to be performed if necessary.

### 4.5.2 Parameter selection

The interface friction ratio, \(\alpha\), required for calculating the frictional resistance in Equation 4.4 is identical to that used in the estimation of frictional resistance on driven
piles and suction caissons and may be estimated as the inverse of the soil sensitivity. For the kaolin considered in this study, the soil sensitivity is 2.5 as quantified using cyclic T-bar penetrometer tests (e.g. Sahdi et al. 2014b, Colreavy et al. 2016), such that $\alpha = 0.4$.

The capacity factor, $N_c$, used in Equation 4.4 for the calculation of bearing resistance for the anchor tip and flukes, is similar to that used in the analysis of driven piles and suction installed caissons. American Petroleum Institute guidelines recommend $N_c = 9$ (API, 2002), although for the ellipsoidal tip considered here, static (nominally undrained) penetration tests suggest that $N_c = 12$ is more appropriate (O’Loughlin et al. 2009). The anchor flukes may be considered similar to the skirt tip of a caisson, which is typically modelled using $N_c = 7.5$, analogous to a deeply embedded strip foundation (Skempton 1951).

The anchor drag coefficient, $C_D$, is geometry dependent and may be assessed experimentally (e.g. Freeman and Hollister 1989; Fernandes et al. 2006; Shelton et al. 2011; Hasanloo et al. 2012; Blake and O’Loughlin 2015) or numerically (Raie et al. 2009; Strum et al. 2010; Nazem et al. 2012). Øye (2000) employed computational fluid dynamics to derive $C_D = 0.63$ for the anchor geometry considered here, which compares well with $C_D = 0.67$ as determined from free-fall in water tests using a 1:20 reduced scale model of the same geometry (O’Beirne et al. 2016). The latter value was adopted in this study.

The reference value, $(v/d)_{ref}$ used in Equation 4.6 is that associated with the measurement of the undrained shear strength, $s_u$. As presented earlier in the paper, $s_u$ was determined from T-bar penetrometer tests involving a T-bar diameter, $d = 5$ mm and a penetration velocity, $v = 1$ mm/s, such that $(v/d)_{ref} = 0.2$ s$^{-1}$, which is four orders of magnitude lower than the maximum value associated with anchor penetration, $v/D = 16.5/0.006 = 2750$ s$^{-1}$.

The strain rate parameter, $\beta$, is best selected based on strain rate dependency observed in variable rate penetrometer tests, as like the case for a dynamically installed anchor, these tests include compensating effects of strain softening that limit the strength increases associated with increasing strain rates. Therefore, $\beta$ values should be guided by those measured in variable rate penetrometer tests, which typically give $\beta = 0.05$ to $0.09$ (Low et al. 2008; Chung et al. 2006), and are approximately equivalent to a 12 to...
21% increase in strength per decade increase in strain rate. A mid-range $\beta = 0.07$ was initially assumed for the embedment prediction model. These model parameters are listed in Table 4.2.

### 4.6 Application to centrifuge test data

As outlined earlier in the paper, continuous motion data captured during dynamic embedment permit for a rigorous assessment of anchor embedment models in accurately describing the processes associated with dynamic penetration. In the following sections, experimental profiles of acceleration and velocity with penetration depth are compared with corresponding theoretical profiles.

Equation 4.2 is appropriate for field cases, but needs to be extended slightly in modelling centrifuge tests to account for the friction between the anchor and the guide, as this represents an additional component of resistance over the first anchor length of penetration (less if the anchor guide is offset from the mudline, as is the case for the centrifuge tests considered here). This was accounted for in the model predictions by including an extra resistance due to the guide friction, equal to $2\mu \omega v_m$ (as discussed earlier in the paper) when the anchor was fully within the guide, but diminishing in a linear fashion as the length of anchor in the guide reduced to zero.

The model was initially applied to Test 2 using the initial parameter set listed in Table 4.2. A comparison of the measured and predicted velocity and acceleration profiles is provided on Figure 4.7, where it is evident that the embedment model over-predicts the final embedment depth, $z_{e,\text{tip}}$, by approximately 20%. Over-prediction of the embedment depth suggests that the resistance forces acting on the anchor during soil penetration are underestimated. O’Loughlin et al. (2009) showed that the parameter set in Table 4.2 is appropriate for modelling the penetration resistance of dynamically anchors under nominally undrained conditions (i.e. excluding the parameters $C_D$ and $\beta$ that account for drag and strain rate effects). As such, the under-prediction of the total penetration resistance – evident from Figure 4.7 – is considered to be due to higher strain rate effects than allowed for using Equation 4.6 and the parameters listed in Table 4.2.

#### 4.6.1 Back calculated strain rate factor

As demonstrated in O’Loughlin et al. (2016), measurement of the anchor acceleration allows the strain rate factor, $R_f$, to be quantified by rearranging Equation 4.2 to give:
\[ R_f = \frac{W - m \frac{d^2 z}{dt^2} - F_b - F_d}{F_{\text{frict}} + F_{\text{bear}}} \]

where \( \frac{d^2 z}{dt^2} \) now represents the linear acceleration derived from the MEMS accelerometer measurements, and \( W, F_b, F_d, F_{\text{frict}} \) and \( F_{\text{bear}} \) are known or formulated quantities using the parameters outlined previously (and listed in Table 4.2). The experimental \( R_f \) determined using Equation 4.8 are shown against velocity on Figure 4.8 for Tests 2, 7 and 15 (refer to Table 4.1), reflecting the range of strength profiles in Figure 4.4. Also shown on Figure 4.8 is \( R_f \) formulated using the power law (Equation 4.6) using \( \beta = 0.07 \) (i.e. as listed in Table 4.2 and used for the prediction on Figure 4.7), \( \beta = 0.13 \) and a varying \( \beta \), discussed further below.

The three tests considered on Figure 4.8 demonstrate a consistent variation in the strain rate factor \( R_f \) with anchor penetration velocity, \( v \), with \( R_f \sim 4 \) at the highest anchor velocity, \( v \sim 17 \text{ m/s} \), where the experimental data appear noisier due to wider variations in the measured acceleration at impact with the mudline and during shallow penetration. \( R_f \sim 4 \), means that the mobilised geotechnical resistance is 4 times higher than would be mobilised under nominal undrained conditions, demonstrating the importance of strain rate effects in dynamic penetration problems. However, as discussed in the following, strain rate effects are more pronounced in these centrifuge tests than in equivalent field conditions.

Also shown on Figure 4.8 is the theoretical variation in \( R_f \) using Equation 4.6, with variations in the magnitude of \( \beta \). O’Loughlin et al. (2013) and Blake and O’Loughlin (2015) suggest that for dynamic penetration problems, \( \beta \) should be similar to that determined from variable rate penetrometer testing, provided that the maximum strain rate in the anchor tests is no more than three orders of magnitude higher than the strain rate associated with the reference soil strength. This is because over several orders of magnitude increase in strain rate, the mobilised shear strength increases more rapidly than can be fitted using a power law (Biscontin & Pestana 2001, Peuchen & Mayne 2007, Lunne & Andersen 2007, Jeong et al. 2009). For these centrifuge tests, where the velocity is unscaled (relative to field conditions), whilst the diameter is scaled by a factor of 100, the maximum (proxy) strain rate is \( v/D = 16.5/0.006 = 2750 \text{ s}^{-1} \), which is four orders of magnitude higher than \( (v/d)_{\text{ref}} = 0.2 \text{ s}^{-1} \). Consequently, and as shown by Figure 4.7, using \( \beta = 0.07 \) with Equation 4.6 significantly underestimates \( R_f \). A higher
operational $\beta = 0.13$ provides a better overall match with the experimentally derived $R_f$, consistent with back analysed $\beta$ values from dynamically installed anchor centrifuge tests reported by O’Loughlin et al. (2013). However, the predictions under-predict strain rate dependency as anchor velocity increases beyond $v \sim 12$ m/s and over-predict strain rate dependency as anchor velocity decreases beneath $v \sim 8$ m/s.

A more reasonable approach would be to allow $\beta$ to retain a lower constant value (within $\beta = 0.06$ to 0.08) over the first two orders of magnitude increase in $v/D$ above the reference $(v/d)_{ref}$, but to then gradually increase with increasing $v/D$. This may be obtained by formulating $\beta$ according to:

$$\beta = \beta_{\text{min}} + \frac{(\beta_{\text{max}} - \beta_{\text{min}})}{1 + \left(\frac{(v/D)_{50}}{v/D}\right)^4}$$

where $\beta_{\text{min}}$ is a lower bound on $\beta$ (in the range 0.06 to 0.08), $\beta_{\text{max}}$ is a limiting strain rate parameter and $(v/D)_{50}$ represents the value of $v/D$ at which $\beta$ is the average of $\beta_{\text{min}}$ and $\beta_{\text{max}}$. The resulting formulated $R_f$ described using Equations 4.8 and 4.9 with $\beta_{\text{min}} = 0.07$, $\beta_{\text{max}} = 0.17$ and $(v/D)_{50} = 1000$ is seen on Figure 4.8 to describe the variation in strain rate dependence adequately, despite slight over-predictions at very low velocities, $v < 1$ m/s.

### 4.6.2 Model predictions

In this section theoretical profiles of acceleration and velocity with penetration depth are compared with corresponding experimental measurements for four tests from the dataset considered here, representing the range of soil strength profiles on Figure 4.4. Modifications were made for each prediction ‘scenario’ to investigate the influence of the resistance components on Figure 4.6. The prediction cases and model parameters are listed in Table 4.3 and investigate scenarios related to: (i) selection of strain rate parameter, (ii) hole closure in the wake of the anchor, (iii) inclusion of drag resistance and (iv) basis for modelling frictional resistance.

The results of these predictions are compared with the corresponding experimental data for the four considered tests in Figures 4.9, 4.10, 4.11 and 4.13 for scenarios (i), (ii), (iii) and (iv) respectively and are discussed in the following sections.
4.6.2.1 Selection of strain rate parameter

The 20% over-prediction of the final anchor embedment depth on Figure 4.7, and back-analysis of R_f on Figure 4.8, suggests that when the strain rate variations exceed three orders of magnitude, soil strength increases much more rapidly than can be described using the power strain rate law, allowing for a typical ~15% increase in soil strength per decade increase in strain rate (β = 0.07 corresponds to a ~15% increase in strength per decade increase in strain rate). The average operable soil strength estimated using β = 0.13 is closer to a 30% increase per log cycle of strain rate, and indeed Figure 4.9 shows that allowing for a higher β (= 0.13; prediction A) leads to final embedments that are within ±1% of the measurements. However, the match between the measured and predicted acceleration profiles improves using β according to Equation 4.9 (prediction B), particularly at deeper embedments where v/D is within three orders of magnitude of (v/d)_{ref} and Equation 4.9 returns β to close to β_{min} = 0.07. For field cases the maximum strain rate is expected to be of the order of 20 to 30 s^{-1}, which is only two orders of magnitude higher than the strain rate associated with in-situ measurement of soil strength using a cone, T-bar or ball penetrometer (v/d = 0.25 to 0.6 s^{-1} for typical penetrometer diameters and a penetration velocity = 20 mm/s). Using Equation 4.9 in this case would maintain β = β_{min} such that Equation 4.9 is unlikely to be required for field conditions. Caution should be exercised if a soil strength measured in laboratory element tests at much lower strain rates (typically 1%/hr) is adopted as the reference soil strength in the power law, as this would result in strain rate variations that span seven orders of magnitude. In such cases the intact laboratory strength should be adjusted to reflect the strain softening associated with anchor installation and the strain rate dependency needs to be more moderate over the first few decades increase in strain rate.

Hole closure

Figure 4.10 compares measured acceleration and velocity profiles with prediction C, which assumes an open hole in the wake of the anchor, and prediction D, which assumes a closed hole. For these tests the maximum anchor embedment depth was 1.47 anchor lengths, such that the difference between modelling an open or closed hole only arises close to the final embedment depth. However, the measured and predicted profiles for z > L are in better agreement when the hole formed by the advancing anchor is modelled according to Equation 4.7, which predicts a closed hole for Tests 2, 4 and 7 and an open hole for Test 15. Although the maximum effect on the final predicted embedment depth is less than ± 2% for the anchor geometry considered here, more
pronounced effects would be expected on objects with lower aspect ratios, such as some designs of free-falling penetrometers for rapid measurement of seabed soil strength (e.g. Stark et al. 2009, Morton et al. 2015).

### 4.6.2.2 Drag resistance

Figure 4.11 shows that the inclusion of drag resistance during soil penetration is appropriate (prediction B), as the agreement between the measured and theoretical profiles is much poorer when it is excluded (prediction E). Attempting to compensate for the lower resistance due to exclusion of drag resistance required an interface friction ratio, $\alpha = 0.51$ to $0.57$ (prediction F), which is incompatible with the remoulded soil strength and still gave a much poorer match to the measurements than was achieved when drag was included and the interface friction ratio was taken as the inverse of the soil sensitivity ($\alpha = 0.4$).

### 4.6.2.3 Frictional resistance

Some studies on the dynamic penetration of cylindrical penetrometers have concluded that strain rates are higher for frictional resistance than for bearing resistance (e.g. Dayal et al. 1975, Steiner et al. 2014, Chow et al. 2014, O’Loughlin et al. 2016). This can be accommodated in a power function by including a coefficient, $n$ (Einav and Randolph, 2006), such that Equation 4.6 becomes:

$$R_f = \left( n \frac{v}{D} \frac{v}{(v/d)_{ref}} \right)^\beta$$  \hspace{1cm} 4.10

Bearing resistance is modelled using $n = 1$ (Zhu and Randolph, 2011), whereas frictional resistance may be modelled either with similar strain rate dependency by also adopting $n = 1$, or by considering $n$ as a function of $\beta$ (adapted from Einav and Randolph, 2006):

$$n = 2 \left( \frac{n_1}{\beta} + n_1 - 2 \right)$$  \hspace{1cm} 4.11

where $n_1 = 1$ for axial loading, i.e. relevant to the ‘axial penetration’ of a dynamically installed anchor. Hence for $\beta = 0.07$ (typical for field conditions) strain rate enhancement of frictional resistance would be approximately 27 times higher than for bearing resistance.
Similar to the methodology adopted for Figure 4.8, strain rate factors for bearing and frictional resistance – $R_{f,\text{bear}}$ and $R_{f,\text{frict}}$ respectively – were determined according to:

\[
R_{f,\text{bear}} = \frac{W - m\frac{d^2z}{dt^2} - F_b - F_d}{n^\beta F_{frict} + F_{\text{bear}}} \quad 4.12
\]

\[
R_{f,\text{frict}} = n^\beta R_{f,\text{bear}} \quad 4.13
\]

where $\beta$ was allowed to vary with $v/D$ according to Equation 4.9. $R_{f,\text{bear}}$ and $R_{f,\text{frict}}$ quantified using Equations 4.12 and 4.13 are shown against velocity on Figure 4.12, together with the theoretical variation in $R_f$ according to Equation 4.11. The agreement between the experimental data and Equation 4.10 shown on Figure 4.12a is clearly quite poor, suggesting that higher strain rate dependency on friction resistance is not warranted. However, a second set of comparisons shown on Figure 4.12b, which were obtained by lowering the interface friction ratio from $\alpha = 0.4$ (Figure 12a) to $\alpha = 0.29$, produces agreement that is equivalent to that obtained using $\alpha = 0.4$, but assuming the same strain rate dependency for friction and bearing (see Figure 4.9). This is made clearer by the predicted and experimental anchor motion profiles on Figure 4.13. Figure 4.13 compares predictions obtained assuming higher strain rate dependency for frictional resistance, with $\alpha = 0.4$ (prediction G) and with $\alpha = 0.29$ (prediction H), with the reference predictions that were obtained assuming the same strain rate dependency for friction and bearing and $\alpha = 0.4$ (prediction B). As expected from Figure 12a, prediction G is in poor agreement with the measurements. Predictions B and H are in very good agreement with the measurements and are essentially coincident.

To explore this further, the embedment model using prediction scenarios B and H was applied to a large database formed from the tests considered here (and listed in Table 4.1) and tests reported by O’Loughlin et al. (2013). The O’Loughlin et al. (2013) data relate to centrifuge tests in normally consolidated clay at 200 g using an anchor with the same geometry as that considered here, but with varying mass and with and without flukes. The comparisons are made on Figure 4.14 in terms of the ratio of predicted to measured final anchor embedment as measurements in the O’Loughlin et al. (2013) tests were limited to the impact velocity and final embedment depth. The initial observation from Figure 4.14 is that the embedment model is capable of predicting the entire database (114 anchor installations) to within ±15% of the measurements, albeit
that \((v/D)_{50} = 5000\) was required for the O’Loughlin et al. (2013) tests, consistent with the slightly lower (on average) reported \(\beta\) values reported in that study. The second observation is that consistent with Figure 4.13, essentially identical predictions are obtained using prediction B (same strain rate dependency for friction and bearing and \(\alpha = 0.4\)) and prediction H (higher strain rate dependency for frictional resistance (Equations 4.10 and 4.11) and \(\alpha = 0.29\)). Finally, Figure 4.14 shows that the prediction accuracy does not exhibit any bias with average strain rate, as quantified by the horizontal axis of Figure 4.14, \((v/D)_{av}\).

A reduced \(\alpha\) is quite reasonable if water becomes entrained at the anchor-soil interface, as is quite possible for dynamic penetration problems. This has also been observed in high-rate ring-shear tests (Tika and Hutchinson, 1999) where water was permitted to penetrate the shear zone, in suction caisson installations where water became trapped between internal stiffeners (Gaudin et al. 2014) and during repeated cyclic penetration of risers from above the mudline to within the seabed (Yuan et al. 2016). Although the data considered here are inconclusive as to the question as to whether strain rate effects are higher for frictional resistance, experimental data on free-fall cone penetrometers instrumented with both tip and shaft load cells are persuasive in this regard (Steiner et al. 2014). This suggests that an allowance for higher strain rate dependence on frictional resistance may be justified, albeit that in some instances the increased frictional resistance may be compensated by a reduced mobilised remoulded soil strength associated with water entrainment.

### 4.7 Conclusions

Dynamically installed anchors may be a cost effective technology for mooring floating facilities, particularly mobile operations, but also risers and pipelines. Predicting their capacity is relatively straightforward (e.g. O’Loughlin et al. 2004a, O’Beirne et al. 2015b) provided their embedment depth can be predicted reliably. This is challenging as it requires an assessment of soil shearing at extremely high strain rates, consideration of drag resistance and other potential hydrodynamic effects such as entrainment of water along a boundary layer at the anchor wall. This paper considers the installation problem by making comparisons of anchor motion data obtained from centrifuge tests in normally and over consolidated clay with predicted responses, as obtained using an analytical embedment model cast in terms of strain rate enhanced shear resistance and drag resistance.
Comparisons of the measured and predicted anchor motion response reveal that the embedment model is capable of describing the anchor motion accurately in both normally and over consolidated clay. Scaling the undrained soil strength to account for the very high strain rates and inclusion of drag resistance were seen to be important considerations. The scaling in the centrifuge tests resulted in strain rates that were about four orders of magnitude higher than the strain rate associated with the measurement of the reference undrained shear strength, and about two orders of magnitude higher than equivalent strain rates in the field. The wider range of strain rates in the centrifuge tests revealed a limitation in the power strain rate law that was adopted here for scaling the soil strength according to the strain rate (although an equivalent limitation would also apply to the semi-logarithmic law). This required the strain rate parameter in the power law to evolve with the mobilised strain rate and a simple backbone curve was proposed in the paper to cater for this.

Predictions of a database of over 100 anchor installations – comprising the centrifuge tests reported in this paper and also centrifuge tests reported by O’Loughlin et al. (2013) – resulted in an accuracy in the final anchor embedment depth that was within ±15% of the measurements. These predictions also revealed that there may be a higher strain rate dependency on frictional resistance than on bearing resistance, although this required the adoption of a reference soil strength that was lower than the fully remoulded strength, suggesting the entrainment of water at the anchor-soil interface.
### 4.8 Tables

#### Table 4.1 Summary of test results

<table>
<thead>
<tr>
<th>Test</th>
<th>Undrained shear strength</th>
<th>Gradient with depth, k (kPa/m)</th>
<th>Impact velocity, v&lt;sub&gt;i&lt;/sub&gt; (m/s)</th>
<th>Tip embedment depth, z&lt;sub&gt;e,tip&lt;/sub&gt; (mm)</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>0.89</td>
<td>16.50&lt;sup&gt;1&lt;/sup&gt;</td>
<td>109.0</td>
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<tr>
<td>2</td>
<td>0</td>
<td>0.89</td>
<td>16.41</td>
<td>110.0</td>
</tr>
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<td>3</td>
<td>0</td>
<td>0.89</td>
<td>16.47</td>
<td>110.5</td>
</tr>
<tr>
<td>4</td>
<td>1.2</td>
<td>1.08</td>
<td>16.55</td>
<td>100.5</td>
</tr>
<tr>
<td>5</td>
<td>1.2</td>
<td>1.08</td>
<td>16.48</td>
<td>99.5</td>
</tr>
<tr>
<td>6</td>
<td>1.2</td>
<td>1.08</td>
<td>16.73</td>
<td>100.0</td>
</tr>
<tr>
<td>7</td>
<td>1.9</td>
<td>1.02</td>
<td>16.92</td>
<td>95.0</td>
</tr>
<tr>
<td>8</td>
<td>1.9</td>
<td>1.02</td>
<td>16.49</td>
<td>96.0</td>
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<tr>
<td>9</td>
<td>2.5</td>
<td>1.08</td>
<td>16.79</td>
<td>92.5</td>
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<td>10</td>
<td>2.8</td>
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<td>16.84</td>
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<td>11</td>
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<td>89.0</td>
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<tr>
<td>12</td>
<td>3.5</td>
<td>1.12</td>
<td>16.57</td>
<td>86.0</td>
</tr>
<tr>
<td>13</td>
<td>3.5</td>
<td>1.12</td>
<td>16.93</td>
<td>86.0</td>
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<tr>
<td>14</td>
<td>4.1</td>
<td>1.15</td>
<td>16.60</td>
<td>85.0</td>
</tr>
<tr>
<td>15</td>
<td>4.1</td>
<td>1.15</td>
<td>16.76</td>
<td>85.0</td>
</tr>
</tbody>
</table>

<sup>1</sup> Estimated value from PERPs data – MEMS accelerometer data not captured
## Table 4.2 Model parameters adopted for initial predictions

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Undrained shear strength, $s_u$ (kPa)</td>
<td>Table 4.1</td>
</tr>
<tr>
<td>Interface friction ratio, $\alpha$</td>
<td>0.4</td>
</tr>
<tr>
<td>Strain rate parameter, $\beta$</td>
<td>0.07</td>
</tr>
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<td>Anchor drag coefficient, $C_D$</td>
<td>0.67</td>
</tr>
<tr>
<td>Bearing capacity factor, $N_c$</td>
<td></td>
</tr>
<tr>
<td>Tip</td>
<td>12</td>
</tr>
<tr>
<td>Fluke</td>
<td>7.5</td>
</tr>
<tr>
<td>Reference strain rate, $(v/d)_{ref}$ (s$^{-1}$)</td>
<td>0.2</td>
</tr>
<tr>
<td>Hole closure</td>
<td>Equation 4.7</td>
</tr>
</tbody>
</table>

## Table 4.3 Model parameters adopted in the various prediction scenarios (modifications to the values/approach given by Table 4.2)

<table>
<thead>
<tr>
<th>Prediction scenario</th>
<th>Prediction</th>
<th>$\beta$</th>
<th>$\beta_{\min}$</th>
<th>$\beta_{\max}$</th>
<th>$(v/D)_{so}$</th>
<th>$n$</th>
<th>$n_l$</th>
<th>Hole closure</th>
<th>$C_D$</th>
<th>$\alpha$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(i) Selection of strain rate parameter (Figure 9)</td>
<td>A</td>
<td>0.13</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Equation 7</td>
<td>0.67</td>
<td>0.4</td>
</tr>
<tr>
<td>(i) Modelling hole closure (Figure 10)</td>
<td>B</td>
<td>Equation 9</td>
<td>0.07</td>
<td>0.17</td>
<td>1000</td>
<td>-</td>
<td>-</td>
<td>Equation 7</td>
<td>0.67</td>
<td>0.4</td>
</tr>
<tr>
<td>(ii) Inclusion of drag resistance (Figure 11)</td>
<td>C</td>
<td>Equation 9</td>
<td>0.07</td>
<td>0.17</td>
<td>1000</td>
<td>-</td>
<td>-</td>
<td>Open</td>
<td>0.67</td>
<td>0.4</td>
</tr>
<tr>
<td>(iii) Modelling frictional resistance (Figure 13)</td>
<td>D</td>
<td>Equation 9</td>
<td>0.07</td>
<td>0.17</td>
<td>1000</td>
<td>-</td>
<td>-</td>
<td>Closed</td>
<td>0.67</td>
<td>0.4</td>
</tr>
<tr>
<td>(iv) Inclusion of drag resistance (Figure 11)</td>
<td>E</td>
<td>Equation 9</td>
<td>0.07</td>
<td>0.17</td>
<td>1000</td>
<td>-</td>
<td>-</td>
<td>Equation 7</td>
<td>0</td>
<td>0.4</td>
</tr>
<tr>
<td>(v) Modelling frictional resistance (Figure 13)</td>
<td>F</td>
<td>Equation 9</td>
<td>0.07</td>
<td>0.17</td>
<td>1000</td>
<td>-</td>
<td>-</td>
<td>Equation 7</td>
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<td>0.51-0.57</td>
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<tr>
<td>(vi) Modelling frictional resistance (Figure 13)</td>
<td>G</td>
<td>Equation 9</td>
<td>0.07</td>
<td>0.17</td>
<td>1000</td>
<td>Equation 11</td>
<td>1</td>
<td>Equation 7</td>
<td>0.67</td>
<td>0.4</td>
</tr>
<tr>
<td>(vii) Modelling frictional resistance (Figure 13)</td>
<td>H</td>
<td>Equation 9</td>
<td>0.07</td>
<td>0.17</td>
<td>1000</td>
<td>Equation 11</td>
<td>1</td>
<td>Equation 7</td>
<td>0.67</td>
<td>0.29</td>
</tr>
</tbody>
</table>
4.9 Figures

Figure 4.1 Dynamically installed anchors: (a) Deep Penetrating Anchor (Deep Sea Anchors 2009), (b) installation procedure (after Lieng et al. 2000)
Figure 4.2 Dynamically installed model anchor featuring an elliptical tip and four large flukes (modelled on an equivalent prototype anchor; Lieng et al. 1999)
Figure 4.3 Centrifuge testing arrangement
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(a) Undrained shear strength, \( s_u \) (kPa)

(b) Undrained shear strength, \( s_u \) (kPa)

(c) Undrained shear strength, \( s_u \) (kPa)

(d) Undrained shear strength, \( s_u \) (kPa)
Figure 4.4 Undrained shear strength and overconsolidation ratio profiles for scrape amounts ($\Delta z_{\text{scrape}}$) of: (a) 0 mm, (b) 15 mm, (c) 25 mm, (d) 35 mm, (e) 45 mm, (f) 55 mm
Figure 4.5 Interpretation of the MEMS accelerometer data: velocity profile during descent through guide and embedment in soil for: (a) Test 2, $s_{um} = 0$ kPa (b) Test 15, $s_{um} = 4.1$ kPa
Figure 4.6 Resistance forces acting on a dynamically installed anchor during installation: option (i) for an open hole in the wake of the anchor and option (ii) for a closed hole in the wake of the anchor.
Figure 4.7 Comparison of experimental and predicted velocity and linear acceleration profiles for Test 2 using the model parameters given in Table 4.2
Figure 4.8 Assessing the performance of the power law in describing the experimentally quantified strain rate dependence
Figure 4.9 Evaluating the merit of allowing the strain rate parameter, $\beta$, to evolve with mobilised strain rate relative to an average ‘operable’ $\beta$: (a) Test 2, (b) Test 4, (c) Test 7 and (d) Test 15
Figure 4.10 Examining the influence of modelling the hole formed in the wake of the anchor as fully closed or fully open: (a) Test 2, (b) Test 4, (c) Test 7 and (d) Test 15
Figure 4.11 Assessing the influence of drag resistance on the anchor motion response: (a) Test 2, (b) Test 4, (c) Test 7 and (d) Test 15
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(a)
Figure 4.12 Evaluation of the power law with allowance for higher strain rate effects on frictional resistance (Equations 4.10 and 4.11) assuming: (a) $\alpha = 0.4$ (b) $\alpha = 0.29$
Figure 4.13 Examining the influence of higher strain rate dependency for frictional resistance: (a) Test 2, (b) Test 4, (c) Test 7 and (d) Test 15
Figure 4.14 Predicted against measured final anchor embedment depths assuming the same strain rate dependency for frictional and bearing resistance with $\alpha = 0.4$ (Prediction B) and higher strain rate dependency for frictional resistance with $\alpha = 0.29$.
Chapter 5

In situ measurement of the dynamic penetration of free-fall projectiles in soft soils using a low-cost inertial measurement unit

**Chapter context:** The paper presented in this chapter describes the technical development of an inertial measurement unit (IMU), crucial for the correct analyses of the field testing database (sub-objective 1). A comprehensive framework for interpreting the motion data recorded by the device is described and validated against direct measurements taken from a range of field tests on two different types of dynamically installed anchor (one being from sub-objective 1) and a free-fall sphere penetrometer. The framework is then used to help calibrate a new analytical embedment model (addressing sub-objective 3) in Chapter 6.

This chapter has been published as:

Main objective: Develop calibrated and validated design tools for predicting the performance of dynamically installed anchors in soft soil

Sub-objective 1: Develop an experimental database, encompassing both centrifuge and field tests, for dynamically installed anchors as a means of establishing anchor performance in soft soil and for calibrating design tools.

Sub-objective 2: Answer the question as to how the soil responds in the wake of a dynamically installed anchor during dynamic embedment, establishing the conditions under which the hole formed by the passage of the anchor may close.

Sub-objective 3: Refine and calibrate analytical embedment models for dynamically installed anchors using motion data collected in the centrifuge and field tests from sub-objective 1.

Sub-objective 4: Establish design tools for predicting the monotonic capacity of dynamically installed anchors for a range of load inclinations, calibrated using field data from sub-objective 1.

Completion of the above sub-objectives leads to the completion of the main objective and Chapter 8.

Chapter 3: Soil response in the wake of dynamically installed projectiles.

Chapter 4: Assessing the penetration resistance acting on a dynamically installed anchor in normally and over consolidated clay.

Chapter 5: In situ measurement of the dynamic penetration of free-fall projectiles in soft soils using a low-cost inertial measurement unit.

Chapter 6: A release-to-rest model for dynamically installed anchors.

Chapter 7: Capacity of dynamically installed anchors as assessed through field testing and three dimensional large deformation finite element analyses.

Chapter 8: Conclusions derived from the work described in the thesis.
5.1 Abstract

Six degree-of-freedom motion data from projectiles free-falling through water and embedding in soft soil are measured using a low-cost inertial measurement unit, consisting of a tri-axial accelerometer and a three-component gyroscope. A comprehensive framework for interpreting the measured data is described and the merit of this framework is demonstrated by considering sample test data for free-falling projectiles that gain velocity as they fall through water and self-embed in the underlying soft clay. The paper shows the importance of considering such motion data from an appropriate reference frame by showing good agreement in embedment depth data derived from the motion data with independent direct measurements. Motion data derived from the inertial measurement unit are used to calibrate a predictive model for calculating the final embedment depth of a dynamically installed anchor.
5.2 Introduction

An inertial measurement unit (IMU) is an electromechanical device that measures an object’s six degree of freedom (6DoF) motion in three-dimensional space using a combination of gyroscope and accelerometer sensors. The development of micro-electro-mechanical systems (MEMS) gyroscope and accelerometer technology has significantly reduced the cost, size, weight and power consumption of IMUs, and enhanced their robustness.

MEMS accelerometers and gyroscopes are typically fabricated on single-crystal silicon wafers using micromachining to etch defined patterns on a silicon substrate. These patterns take the form of small proof masses that are free from the substrate and surrounded by fixed plates. The proof mass is connected to a fixed frame by flexible beams, effectively forming spring elements. Low-cost consumer grade MEMS gyroscopes typically use vibrating mechanical elements to sense angular rotation rate. During operation the proof mass is resonated with constant amplitude in the ‘drive direction’ by an external sinusoidal electrostatic or electromagnetic force. Angular rotation then induces a matched-frequency sinusoidal Coriolis force orthogonal to the drive-mode oscillation and the axis of rotation. The Coriolis force deflects the proof mass and plates connected to the proof mass move between the fixed plates in the sense-mode.

The operational principle for MEMS accelerometers is much simpler; accelerations acting on the proof mass cause it to displace, and plates connected to the proof mass move between fixed plates. For both sensors, the movement of the plates cause a differential capacitance that is measured by integrated electronics and is output as a voltage that is proportional to either the applied angular rotation rate (in the case of MEMS gyroscopes) or acceleration (in the case of MEMS accelerometers). The operational principles of the MEMS accelerometers and gyroscopes as described above are shown schematically in Figure 5.1.

Common applications of low-cost IMUs featuring MEMS technology include: inertial navigation systems (e.g. remotely operated vehicles, autonomous underwater vehicles and unmanned aerial vehicles), active safety systems (electronic stability control and traction control in motor vehicles) and motion-activated user-interfaces (e.g. smartphones, game controllers and tablet computers). The use of low-cost 6DoF IMUs for geotechnical applications has not been reported. However, MEMS accelerometers have been used for in situ geotechnical applications to measure: inclinations in boreholes (Bennett et al. 2009), soil displacement associated with rapid uplift of
footings (Levy and Richards 2012) and the motion of free-falling cone penetrometers (e.g. Stegmann et al. 2006; Stephan et al. 2012; Steiner et al. 2014). In geotechnical centrifuge modelling MEMS accelerometers have been used to measure: the acceleration response of free-falling projectiles in clay (O’Loughlin et al. 2014; Chow et al. 2014), earthquake accelerations (Cilingir and Madabhushi 2011; Stringer et al. 2010) and rotation of structures during slow lateral cycling and dynamic shaking (Allmond et al. 2014). Although accelerometers are often used to measure the rotation of objects at constant acceleration, they cannot distinguish rotation from linear acceleration if the object’s orientation and acceleration is changing. However, gyroscopes are unaffected by linear acceleration, and the rotation of accelerating objects can be derived from their measurements. Hence the combination of accelerometer and gyroscope measurements enables an object’s linear acceleration to be determined relative to a reference frame that is not necessarily coincident with the reference frame of the object. This becomes important for the applications considered in this paper, where dynamically installed anchors and a free-falling sphere (collectively referred to as ‘projectiles’ from this point forward) free fall through water and bury in the underlying soil. As described later, the motion response of the projectile must be considered from the appropriate reference frame. From the viewpoint of the hydrodynamic and geotechnical resistances acting on the projectile during motion, it becomes important to consider the projectile’s trajectory, whereas from a geotechnical design viewpoint the final depth and orientation of the projectile relative to a fixed inertial frame of reference (with an axis in the direction of Earth’s gravity) is important as this will dictate the local soil strength in the vicinity of the embedded projectile and (for the case of the anchors) how this strength will be mobilised during loading.

This paper describes a custom-design, low-cost MEMS based IMU and presents a comprehensive framework for interpreting the IMU measurements (which are made in the body frame of reference) so that they are coincident with a fixed inertial frame of reference. The framework is implemented to establish rotation, acceleration and velocity profiles for the projectiles during free-fall in water and embedment in soil. The final projectile embedment depths established from the IMU data are compared with direct measurements, and the merit of collecting motion data during dynamic penetration is demonstrated by using such data to verify the appropriateness of an embedment prediction model for dynamically installed anchors.
5.3 Free-falling projectiles

5.3.1 Deep Penetrating Anchors

The deep penetrating anchor (DPA) is a proprietary term for a dynamically-installed anchor design. The DPA is designed so that, after release from a designated height above the seafloor, it will penetrate to a target depth in the seabed using the kinetic energy gained through free-fall. The DPA data considered here are from tests using a 1:20 reduced scale model anchor based on an idealised design proposed by Lieng et al. (1999). The model DPA (see Figure 5.2), was fabricated from mild steel and had an overall length of 750 mm, a shaft diameter of 60 mm and a mass of 20.7 kg. The anchor had an ellipsoidal tip and featured four clipped delta type flukes (separated by 90° in plan) with a forward swept trailing edge. The anchor shaft was solid with the exception of a watertight cylindrical void towards the top to house the IMU.

5.3.2 Dynamically Embedded Plate Anchors

The dynamically embedded plate anchor (DEPLA, O’Loughlin et al. 2013a) is an anchoring system that combines the capacity advantages of vertically loaded anchors with the installation advantages of dynamically installed anchors. The DEPLA comprises a removable central shaft or ‘follower’ and a set of four flukes (see Figure 5.3). A stop cap at the upper end of the follower prevents it from falling through the DEPLA sleeve and a shear pin connects the flukes to the follower. The DEPLA is installed in a similar manner as the DPA, but after coming to rest in the seabed the follower retriever line is tensioned, which causes the shear pin to part (if not already broken during impact) allowing the follower to be retrieved for the next installation whilst leaving the anchor flukes vertically embedded in the seabed. These embedded anchor flukes constitute the load bearing element as a plate anchor.

In the tests considered here the DEPLA was modelled at a reduced scale of 1:4.5 and fabricated from mild steel. The follower (and hence DEPLA) length was 2 m, the follower diameter was 160 mm, the fluke (plate) diameter was 800 mm and the overall mass was 388.6 kg. As with the DPA, the DEPLA follower was solid with the exception of a cylindrical void at the top to house the IMU. The model DEPLA is shown in Figure 5.3.
5.3.3 Instrumented Free-Falling Sphere

The instrumented free-falling sphere (IFFS) has been proposed as an in-situ characterisation tool for soft soils (Morton and O’Loughlin 2012; O’Loughlin et al. 2014). The IFFS is a steel sphere that dynamically embeds in soft soil in a manner similar to dynamically installed anchors. IMU data measured during embedment in soil can be used to estimate undrained shear strength. As such, the IFFS is conceptually similar to a free fall cone penetrometer, but the simple spherical geometry of the IFFS is beneficial as the projected area does not change with rotation and the bearing factor for the ball is more tightly constrained than for the cone. The IFFS data considered here are from tests using a 250 mm diameter mild steel sphere with a mass of 50.8 kg. The IFFS was fabricated as two hemispheres (that could be bolted together) with an internal vertically orientated cylindrical void to accommodate the IMU (see Figure 5.4).

5.4 Inertial Measurement Unit

The IMU was used to measure projectile accelerations and rotation rates during free-fall in the water column and embedment in the soil. The IMU (see Figure 5.5) includes a 16 bit three component MEMS rate gyroscope (ITG 3200) and a 13 bit three-axis MEMS accelerometer (ADXL 345). The gyroscope had a resolution of 0.07 °/s with a measurement range of +/- 2000 °/s. The accelerometer had a resolution of 0.04 m/s² with a measurement range of +/- 16 g. Data were logged by an mbed micro controller with an ARM processor to a 2 GB SD card at 400 Hz. Internal batteries were capable of powering the logger for up to 4 hours. The IMU was contained in a watertight aluminium tube 185 mm long and 42 mm in diameter and was located in a void (with the same dimensions) within the projectile. The IMU had a mass of approximately 0.5 kg (including the batteries).

The accelerometer and gyroscope are aligned with the body frame of the projectile and the IMU as shown in Figure 5.6 (for the DEPLA). The body frame is a reference frame with three orthogonal axes $x_b, y_b$ and $z_b$ that are common to both the IMU and the projectile and where the $z_b$-axis is parallel to the direction of earth’s gravity when the projectile is hanging vertically. The accelerometer measures accelerations $A_{bx}, A_{by}$ and $A_{bz}$ in the body frame along these three axes. These accelerometer measurements include a component of gravitational acceleration (depending on the orientation of the accelerometer) and linear acceleration. The gyroscope measures angular velocities $\omega_{bx}, \omega_{by}$ and $\omega_{bz}$ in the body frame about the same orthogonal axes. Accelerometers are often
used to measure the rotation of quasi-static objects but cannot distinguish rotation from linear acceleration if an object is in motion. However, gyroscopes are unaffected by linear acceleration and the rotation of objects in motion can be derived from their measurements.

5.5 Interpretation of IMU Measurements

As the body frame is not fixed in space, it is necessary to define an inertial frame, defined here and used in this paper, as a local fixed reference frame, with the z-axis aligned in the direction of the Earth’s gravitational vector, and with undefined orthogonal x- and y-axes, that are fixed at their orientation at the start of each test. If the projectile pitches and/or rolls whilst in motion, the body frame will move out of alignment with the inertial frame of reference and the rotation rates \( \omega_{bx}, \omega_{by}, \omega_{bz} \) and accelerations \( A_{bx}, A_{by}, A_{bz} \) measured by the IMU will not be coincident with the inertial frame (see Figure 5.7). As a consequence gravitational acceleration \( g \), and linear acceleration \( a \) (required for velocity and translation calculations as described later), components cannot be distinguished from the accelerometer measurements. Hence the IMU measurements were ‘transformed’ from the body frame to the inertial frame. This was accomplished using transformation matrices as described in the following sections.

5.5.1 Rotation

The body frame rotation rates \( \omega_{bx}, \omega_{by}, \omega_{bz} \) were transformed from the body frame to the inertial frame to correspond with rotation rates about the inertial frame \( \omega_x, \omega_y, \omega_z \) using an angular velocity transformation matrix (AVTM), \( T_b^i \) (Fossen 2011):

\[
\begin{pmatrix}
\omega_x \\
\omega_y \\
\omega_z
\end{pmatrix} = T_b^i \begin{pmatrix}
\omega_{bx} \\
\omega_{by} \\
\omega_{bz}
\end{pmatrix}
\]

\[
T_b^i = \begin{bmatrix}
1 & \sin(\phi_b)\tan(\theta_b) & \cos(\phi_b)\tan(\theta_b) \\
0 & \cos(\phi_b) & -\sin(\phi_b) \\
0 & \sin(\phi_b) / \cos(\theta_b) & \cos(\phi_b) / \cos(\theta_b)
\end{bmatrix}
\]

where \( \phi_b \) and \( \theta_b \) are the current rotation angles about the body frame axes \( x_b \) and \( y_b \) respectively established from numerical integration of \( \omega_{bx} \) and \( \omega_{by} \):
5.5.2 Acceleration

The accelerometer measurements $A_{bx}$, $A_{by}$ and $A_{bz}$ were converted to accelerations coincident with the inertial frame $A_x$, $A_y$ and $A_z$ using a direction cosine matrix (DCM) $R'_b$ (Nebot and Durrant-Whyte 1999; Jonkman 2007; King et al. 2008; Fossen 2011):

$$
\begin{pmatrix}
A_x \\
A_y \\
A_z
\end{pmatrix} =
R'_b
\begin{pmatrix}
A_{bx} \\
A_{by} \\
A_{bz}
\end{pmatrix}
$$

$$
R'_b = R_z(-\psi)R_y(-\theta)R_x(-\phi)
$$

The DCM relates the accelerations measured in the body frame to the inertial frame by considering three successive rotations of yaw $-\psi$, pitch $-\theta$, and roll $-\phi$, about the inertial frame axes $z$, $y$ and $x$ respectively. These rotations are represented by the yaw $R_z(-\psi)$, ...
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The linear accelerations coincident with the inertial frame \( a_x \), \( a_y \) and \( a_z \) were derived from the transformed accelerometer measurements \( A_x \), \( A_y \) and \( A_z \) (\( A_z \) is a negative output, i.e. when the projectile is at rest, \( a_z = A_z + g = 0 \)) using the following expression: (Stovall 1997; Noureldin et al. 2012):

\[
\begin{align*}
\begin{bmatrix}
  a_x \\
  a_y \\
  a_z 
\end{bmatrix}
  &=
\begin{bmatrix}
  A_x \\
  A_y \\
  A_z 
\end{bmatrix}
+ 
\begin{bmatrix}
  0 \\
  0 \\
  g 
\end{bmatrix}
\end{align*}
\]

5.15

The resultant linear acceleration, \( a \) (acceleration in the direction of motion), was calculated as:

\[
a = \sqrt{A_x^2 + A_y^2 + A_z^2} - g
\]

5.16

5.5.3 Velocity and Distance

The linear accelerations corresponding to the inertial frame \( a_x \), \( a_y \) and \( a_z \) were numerically integrated to establish the projectile velocities coincident with the inertial frame \( v_x \), \( v_y \) and \( v_z \) during free-fall in the water column and embedment in the soil:
The resultant projectile velocity \( v \), was calculated using the following expression:

\[
v = \sqrt{v_x^2 + v_y^2 + v_z^2}
\]

The resultant projectile velocity \( v \), was numerically integrated to establish the distance travelled by the projectile along its trajectory \( s \):

\[
s(t) = s_0 + \int_0^t v(t)\,dt
\]

The distance travelled by the projectile along the inertial \( z \) axis \( s_z \) (required to calculate the vertical embedment depth of the projectile relative to the soil surface, \( z_e \)), was established by numerically integrating the vertical velocity \( v_z \):

\[
s_z(t) = s_{z0} + \int_0^t v_z(t)\,dt
\]

### 5.5.4 Tilt Angles

Following dynamic penetration the projectile is at rest in the soil and has no linear acceleration. Under these conditions the accelerometer measurements can be used to derive the final pitch \( \phi_{acc} \), roll \( \theta_{acc} \), (coincident with the inertial frame) and resultant tilt \( \mu \), (tilt relative to Earth’s gravitational vector, see Figure 5.7) angles using the following expressions:

\[
\phi_{acc} = \sin^{-1} \left( \frac{A_{by}}{g} \right) \quad \text{(King et al. 2008)}
\]

\[
\theta_{acc} = \sin^{-1} \left( \frac{A_{bx}}{g} \right) \quad \text{(King et al. 2008)}
\]
\[ \mu = \cos^{-1} \left( \frac{A_b}{g} \right) \] (Stephan et al. 2012)

5.6 Test Sites and Soil Properties

The IMU performance has been examined using projectile data from two sites. The DEPLA data considered here relate to tests conducted in the Firth of Clyde which is located off the West coast of Scotland between the mainland and the Isle of Cumbrae. The DPA and IFFS data are from tests conducted in Lower Lough Erne, which is an inland lake located in County Fermanagh, Northern Ireland. At Lough Erne the water depths at the test locations varied between 3 and 20 m whereas at the Firth of Clyde test locations the water depth was typically 50 m. Both test locations are shown in Figure 5.8.

The seabed at the DEPLA test locations in the Firth of Clyde is very soft with moisture contents in the range 50 to 100% (close to the liquid limit). Consistency limits plot above or on the A-line on the Casagrande plasticity chart, indicating a clay of intermediate to high plasticity. The unit weight increases from about \( \gamma = 14 \text{kN/m}^3 \) at the mudline to about \( \gamma = 18 \text{kN/m}^3 \) at about 3.5 m (limit of the sampling depth). Figure 5.9a shows profiles of undrained shear strength \( s_u \), with depth derived from piezocone and piezoball tests, and calibrated using lab shear vane data and fall cone tests, to give piezocone bearing factors \( N_{kt} = 17.8 \) (5 cm\(^2\) cone) and \( N_{kt} = 16.9 \) (10 cm\(^2\) cone), and piezoball bearing factors \( N_{ball} = 11.5 \) (50 cm\(^2\) ball) and \( N_{ball} = 12.2 \) (100 cm\(^2\) ball). The \( s_u \) profile is best idealised as \( s_u \) (kPa) = 2 + 2.8z over the upper \( z = 5 \) m of the penetration profile, which is the depth of interest for the DEPLA tests. The ratio of remoulded to intact soil resistance is in the range 0.19 to 0.33 as assessed from piezoball cyclic remoulding tests. This range is similar, but not identical to the range of soil sensitivity, as the bearing factor for remoulded soil is greater than for intact soil (Yafrate et al. 2009; Zhou and Randolph 2009).

The Lough Erne lakebed is very soft clay with moisture contents in the range 270 to 520%, typically about 1.5 times the liquid limit. The measured unit weight of the Lough Erne clay is only marginally higher than water at \( \gamma = 10.8 \text{kN/m}^3 \). This is considered to be due to the very high proportion of diatoms that are evident from scanning electron microscopic images of the soil (e.g. see Colreavy et al. 2012) and which have an enormous capacity to hold water in the intraskeletal pore space (Tanaka and Locat
1999). Colreavy et al. (2012) report data from piezoball penetration tests (using a 100 cm$^2$ ball) at the Lough Erne site to depths of up to 8 m. Figure 5.9b shows $s_u$ profiles with depth, obtained from the net penetration resistance using $N_{ball} = 8.6$, calibrated using in-situ shear vane data. The undrained shear strength profile is best idealised over the depth of interest (0 to 2.2 m) as $s_u$ (kPa) = 1.5z. Piezoball cyclic remoulding tests show that the ratio of remoulded to intact soil resistance is in the narrow range 0.4 to 0.5, indicating a low sensitivity soil.

5.6.1 Test procedure

Testing was conducted using the RV Aora, a 22 m research and survey vessel in Firth of Clyde (Figure 5.10a) and either a fixed vessel berthing jetty or a 15 m self-propelled barge (Figure 5.10b) in Lough Erne. The self-propelled barge was equipped with a 13 tonne winch and a 2 tonne crane, whereas the RV Aora was equipped with several winches and an 8 tonne crane. The testing procedure for each site and projectile was broadly similar (summarised schematically in Figure 5.11 for the DEPLA tests using the RV Aora) and involved the following stages:

1. The IMU was powered up and secured in the projectile.
2. The projectile was lowered below the water surface to the desired drop height above the mudline.
3. The projectile was released by opening a quick release shackle connecting the projectile release/retrieval line to the crane, allowing the projectile to free-fall and penetrate the soil.
4. The projectile tip embedment depth $z_e$, was measured by sending a remotely operated vehicle (ROV) (Firth of Clyde), or a drop camera (Lough Erne) to the mudline to inspect markings on the projectile retrieval line (see Figure 5.12).

5.7 Results and Discussion

The IMU data were interpreted within the framework described above, which can be readily implemented in a spreadsheet application such as Microsoft Excel or alternatively using numerical analysis software such as MATLAB.

5.7.1 Rotation

Rate gyroscopes are subject to an error known as bias drift where the zero rate output drifts over time (Sharma 2007). However, the duration of a projectile drop never
exceeded 6.5 s, which is too short for any measurable bias drift to accumulate. This was confirmed by comparing the zero rate outputs before the drop when the anchor was hanging in the water with the zero rate outputs after the drop when the anchor was at rest in the soil. No change was observed for any test.

Figure 5.13 shows typical rotation profiles during free-fall in water and embedment in the lakebed for each of the three projectiles, released from drop heights of 17.69 m (DEPLA), 5.95 m (IFFS), and 3 m (DPA). In Figure 5.13 $\phi_{\text{acc}}$ and $\theta_{\text{acc}}$ are rotations relative to the inertial frame deduced from the horizontally orientated $y$- and $x$-axes accelerometers using Equations 5.23 and 5.24, $\phi_b$, $\theta_b$ and $\psi_b$ are rotations about the body frame axes $x_b$, $y_b$ and $z_b$ established using Equations 5.3, 5.4 and 5.5, and $\phi$, $\theta$ and $\psi$ are the pitch, roll and yaw rotations about the inertial frame axes $x$, $y$ and $z$ derived using Equations 5.6, 5.7 and 5.8.

In Figure 5.13a, prior to release (time, $t = 0$ to 1.1 s) the DEPLA was swaying in the water, suspended from the installation line, during which time rotations derived from the accelerometer measurements ($\phi_{\text{acc}}$ and $\theta_{\text{acc}}$) and from the gyroscope measurements ($\phi_b$ and $\theta_b$) were in broad agreement. During free-fall ($t = 1.1$ s to 3.59 s) rotations can only be interpreted from the gyroscope measurements as the accelerometer measurements include both acceleration and rotation components. The gyroscope measurements indicate that rotations reached $\phi_b = 17.3^\circ$ and $\theta_b = -8.3^\circ$ when the anchor came to rest in the lakebed at $t = 4.2$ s. There is a discrepancy of $\Delta\phi = 1.7^\circ$ and $\Delta\theta = 3.1^\circ$ between the accelerometer and gyroscope measurements whilst the anchor is at rest. However, when the anchor was at rest in the soil the ‘transformed’ rotations derived from the gyroscope measurements ($\phi$ and $\theta$) were in good agreement with rotations derived from the accelerometer measurements, as both were coincident with the inertial frame of reference.

Figure 5.13b shows that the IFFS rotated about all three axes during freefall in water and penetration in soil. Indeed, the non-zero $\psi_b$ and $\psi$ response started whilst the IFFS was hanging in water, indicating that the IFFS started to spin before it was released. After the IFFS came to rest in the soil there is a discrepancy of $\Delta\phi = 4.1^\circ$ and $\Delta\theta = 2.8^\circ$ between the final accelerometer and gyroscope measurements. As with the DEPLA test, the transformed rotations derived from the gyroscope measurements were in good agreement with rotations derived from the accelerometer measurements. This highlights the importance of using the AVTM to transform the angular velocities measured by the
gyroscope from the body frame to the inertial frame to establish rotations that relate to the inertial frame.

In contrast, rotations measured during the DPA free-fall and embedment phases (Figure 5.13c) were much lower than from the DEPLA and IFFS tests. Indeed, the rotation appears to have only occurred before release (due to swaying and spinning in water) and at the start of the free-fall phase, indicating that the DPA tends to self-correct and become hydrodynamically stable during free-fall in water. As such the misalignment between the body frame of the IMU (and hence the anchor) and the inertial frame of reference in this case was negligible, with no discernible differences in the rotations derived from the final accelerometer and gyroscope measurements when the anchor came to rest in the soil. Hence transformation of rotations between the body frame and the inertial frame may not be warranted in cases where the rotations are relatively small.

5.7.2 Acceleration

Figure 5.14 shows acceleration profiles for the same tests as shown in Figure 5.13. In Figure 5.13 $A_{bx}$, $A_{by}$ and $A_{bz}$ are the accelerometer measurements and $A_x$, $A_y$ and $A_z$ are the transformed accelerometer measurements that are coincident with the inertial frame (i.e. $A_z$ is the acceleration measurement in the direction of gravity). In Figure 5.14a the DEPLA was initially hanging in the water experiencing only gravitational acceleration with $A_x = 0$ ($a_x = 0$), $A_y = 0$ ($a_y = 0$) and $A_z = -9.81 \, \text{m/s}^2$ (i.e. $a_z = 0$, refer to Equation 5.15). Following release at $t = 1.1$ s the anchor began to free-fall in water with an abrupt change in $A_z$ to $-0.81 \, \text{m/s}^2$ ($a_z = 9 \, \text{m/s}^2$). From $t = 1.1$ to 3.59 s the anchor was in free-fall through water and $A_z$ (and hence $a_z$) steadily reduced as the fluid drag resistance increased with increasing anchor velocity. Impact with the mudline occurred at $t = 3.59$ s and is characterised by a rapid deceleration to a maximum value of approximately $A_z = -41.6 \, \text{m/s}^2$ ($a_z = -3.2g = -31.8 \, \text{m/s}^2$). The anchor came to rest at $t = 4.2$ s before rebounding slightly. This rebound has been reported in other studies involving free-fall objects (e.g. Dayal and Allen 1973; Chow and Airey 2010; Morton and O’Loughlin 2012; O’Loughlin et al. 2014), and is attributed to elastic rebound of the soil. The importance of transforming the measured accelerations to the inertial frame using the DCM is evident from the soil penetration phase where the magnitude of the peak inertial frame deceleration $A_z$ is 3.7% lower than the peak body frame deceleration $A_{bz}$. Furthermore, when the anchor was at rest the inertial frame accelerations $A_x$ and $A_y$ sensibly returned to zero and $A_z = -9.81 \, \text{m/s}^2$ ($a_z = 0$) in the absence of linear
acceleration, whereas the body frame accelerations, $A_{bx}$ and $A_{by}$ are non-zero, and $A_{bz} \neq -9.81 \text{ m/s}^2$ due to anchor rotations causing misalignment between the body and inertial frames.

The acceleration response of the IFFS (Figure 5.14b) is broadly similar to that of the DEPLA, with the expected change in acceleration upon release and the subsequent reduction in acceleration due to increasing fluid drag resistance. Accelerations also reduce markedly upon impact with the soil surface, although the absolute deceleration is lower than for the DEPLA due to the lower soil strength at this site. The sudden reduction in the accelerations along the z-axis during penetration in soil (evident in both the body frame and the inertial frame accelerations) is considered to be due to changes in the soil flow regime. This influences the magnitude of the drag resistance that dominates at these very shallow embedment depths in very soft soil and at high penetration velocities (Morton et al. 2015).

Figure 5.14c shows the acceleration response for the DPA test. The response is qualitatively similar to those shown in Figure 5.14a and Figure 5.14b for the DEPLA and the IFFS respectively, although there is negligible difference between the body frame accelerations and the transformed inertial frame accelerations as rotations where relatively small for this test.

### 5.7.3 Velocity profiles

Figure 5.15 shows velocity profiles for free-fall in water and embedment into soil for the three tests considered previously and shown in Figures 5.13 and 5.14. The velocity $v_z$, and distance $s_z$, (i.e. depth) relative to the inertial frame were established using Equations 5.19 and 5.20. The velocity $v_{hz}$, and distance $z_{hz}$, were also derived from Equations 5.19 and 5.20, albeit with $a_{hz} = A_{hz} + g$, instead of $a_z$ and $A_z$. $v_{hz}$ and $z_{hz}$ represent the values that would otherwise be used if the IMU measurements were not corrected using the AVTM and DCM. The importance of implementing the transformation matrices is demonstrated in Figure 5.15a where the final embedment depth and impact velocity of the DEPLA are over estimated by 12% and 7% respectively. This would correspond to an over prediction of the local undrained shear strength (and hence capacity) at the mid-height of the DEPLA plate (following installation but prior to keying) of 17% based on the final tip embedment of $z_e = 3.31 \text{ m}$ and the idealised strength profile, $s_u \text{ (kPa)} = 2 + 2.8z$. Figure 5.15b indicates that the embedment depth and impact velocity of the IFFS are over predicted by 27% and 10%
respectively. The over prediction for the IFFS is higher than for the DEPLA as the IFFS rotations are higher (i.e. greater misalignment between the body- and inertial frames). Figures 5.15a and 5.15b also show that the velocity $v_{bcz}$, established from the integration of the body frame ‘linear’ acceleration $a_{bcz}$, does not return to zero despite motion having ceased. This is because the body frame acceleration measurement $A_{bcz}$ (from which $a_{bcz}$ is derived) is not coincident with the inertial frame and does not return to zero following installation (i.e. $A_{bcz} > -9.81 \text{ m/s}^2$). The DPA body frame and inertial frame velocity profiles (Figure 5.15c) are in excellent agreement as the rotations are relatively low and the misalignment between the body frame and inertial frame is negligible. Also shown on Figure 5.15 are direct measurements of the final embedment depths based on mudline observations of markings on the retrieval line using a ROV in Firth of Clyde and an underwater drop camera in Lough Erne. Final embedment depths derived from the IMU data are within 3.3% of the direct measurements, with differences of 0.09 m (DEPLA), 0.06 m (IFFS) and 0.035 m (DPA). However, the direct measurements are simply to confirm the lack of any gross error in the analysis, and have a much lower accuracy than is possible from the IMU data. A more rigorous verification of the IMU derived measurements was undertaken for a number of tests as described in the following section.

### 5.7.4 Verification of the IMU derived measurements

Independent measurement of the projectile displacement was obtained by comparing the IMU derived displacement measurements (Equation 5.22) with those obtained from a draw wire sensor (string potentiometer) with a 10 m measurement range. The draw wire sensor was connected between a fixed point on the deployment platform and the free falling projectile (i.e. connected in parallel with the deployment and retrieval line), and the data acquired using an independent 24-bit data acquisition system. Five tests were undertaken using the IFFS projectile from various drop heights above the lakebed. Comparisons of displacements derived from the IMU measurements and the draw wire sensor data are provided in Figure 5.16. The IMU derived displacements are shown both derived from the body reference frame and the inertial reference frame, where it is evident that the inertial frame displacements sensibly remain constant when the projectile comes to rest in the soil, whereas the body frame derived displacements derived from the body frame IMU data continue to increase as the velocity has not returned to zero (see also Figure 5.15). Importantly, excellent agreement is apparent
between the inertial frame reference frame displacements and those measured by the draw wire sensor (within 1% of the measurement range), providing verification of the analysis approach outlined here.

5.7.5 Example Application of Projectile IMU Data

For the projectiles considered in the previous section, understanding the soil-structure interaction at such high strain rates is crucial for predictive tools that calculate the final embedment depth of the anchors (DEPLA and DPA, e.g. O’Loughlin et al. 2013b) or estimate the undrained shear strength based on the interpreted inertial frame accelerations (IFFS, O’Loughlin et al. 2014; Morton et al. 2015). This is because those strain rates are up to seven orders of magnitude higher than used for strength determination in a standard laboratory element test. It follows that motion data such as those presented in Figures 5.13 and 5.14 play an important role in the validation and calibration of such predictive models. An example comparison is provided in Figure 5.17 for the DEPLA, where the predictions are based on an analytical model described in brief here, but in more detail by O’Loughlin et al. (2013b). The model formulates conventional end bearing and frictional resistance acting on the anchor during penetration in a manner similar to suction caisson or pile installation, but scales these resistances to account for the well-known dependence of undrained shear strength on strain rate (Casagrande and Wilson 1951, Graham et al. 1983, Sheahan et al. 1996), whilst also accounting for drag resistance and the buoyant weight of the displaced soil. Consideration of these resistance components leads to the following governing equation:

\[ m \frac{d^2 s}{dt^2} = W_s - F_b - R_f (F_{frict} + F_{bear}) - F_d \]

where \( m \) is the anchor mass, \( s \) is the distance travelled by the projectile, \( t \) is time, \( W_s \) is the submerged weight of the anchor in water, \( F_{frict} \) is frictional resistance, \( F_{bear} \) is bearing resistance, \( F_b \) is the buoyant weight of the displaced soil and \( F_d \) is drag resistance, formulated as:

\[ F_d = \frac{1}{2} C_d \rho_s A_p v^2 \]

where \( \rho_s \) is the submerged density of the soil, \( C_d \) is the drag coefficient, \( A_p \) is anchor projected (frontal) area and \( v \) is the instantaneous resultant anchor velocity. The
inclusion of drag, $F_d$, is essential in situations where (non-Newtonian) very soft fluidised soil is encountered at the surface of the seabed, and has been shown to be important for assessing loading from a submarine slide runout on a pipeline (Boukpeti et al. 2012, Randolph and White 2012, Sahdi et al. 2014). O’Loughlin et al. (2013) and Blake and O’Loughlin (2015) further showed that drag is the dominant resistance acting on a dynamically installed anchor in normally consolidated clay during initial embedment and typically to about 30% of the penetration.

Frictional and bearing resistances are formulated as

$$F_{\text{frict}} = \alpha s_u A_s; F_{\text{bear}} = N s_u A_p$$

5.28a; b

where $\alpha$ is an interface friction ratio (of limiting shear stress to undrained shear strength), $A_s$ is anchor shaft area, $N$ is the bearing capacity factor for the projectile tip or fluke, and $s_u$ is the undrained shear strength averaged over the contact area, $A_p$ or $A_s$.

The reference undrained shear strength adopted in Equation 5.28 is the idealised profile shown in Figure 5.9a, which is enhanced using a power law strain rate function (Biscontin and Pestena 2001; Peuchan and Mayne 2007; Randolph et al. 2007; O’Loughlin et al. 2013b) expressed as:

$$R_f = \left( \frac{\dot{\gamma}}{\dot{\gamma}_{\text{ref}}} \right)^\beta \approx \left( n \frac{v/d}{(v/d)_{\text{ref}}} \right)^\beta$$

5.29

where $\beta$ is the strain rate parameter, $v/d$ is an approximation of the operational shear strain rate, and the subscript ‘ref’ denotes the reference shear strain rate associated with the measurement of the undrained shear strength. The factor $n$ in Equation 5.29 accounts for the greater rate effects reported for shaft resistance compared to tip resistance (Dayal et al., 1975; Chow et al., 2014; Steiner et al. 2014) and is taken as $n = 1$ for tip resistance (Zhu and Randolph, 2011) and as a function of $\beta$ (adopted from 5.8b in Einav and Randolph, 2006) for estimating rate effects in shaft resistance according to:

$$n = 2 \left( \frac{n_l}{\beta} + n_l - 2 \right)$$

5.30

where $n_l$ is 1 for axial loading.

The predictions on Figure 5.17 were obtained using bearing capacity factors of $N = 7.5$ for the leading and trailing edges of the flukes (analogous to a deeply embedded strip
footing) and $N = 12$ for the follower tip, but not for the padeye as the hole formed by the passage of the anchor was assumed to remain open. This is appropriate since ROV video capture of the drop sites (see Figure 5.13) showed an open crater and the dimensionless strength ratio at the trailing end of the embedded DEPLA follower, $s_u/\gamma'D = 6.9$ (where $D$ is the diameter of the DEPLA sleeve and $\gamma'$ is the effective unit weight of the soil), which is sufficient to maintain an open cavity above the follower (Morton et al. 2014). Values for the drag coefficient, $C_d$, were determined from the free-fall in water phase of the tests, which gave an average $C_d = 0.7$ (Blake and O’Loughlin, 2015).

The strain rate parameter was taken as $\beta = 0.08$, which is typical of that measured in variable rate penetrometer testing (Low et al. 2008, Lehane et al. 2009) and approximates to an 18% change per log cycle change in strain rate, typical of that measured in laboratory testing (e.g. Vade and Campenella 1977, Graham et al. 1983, Lefebvre and Leboeuf 1987). The interface friction ratio, $\alpha$, was varied to obtain the best match between the measured and predicted velocity profiles. The comparison between these on Figure 5.17 indicates that the inclusion of a fluid-mechanics drag resistance term is appropriate for projectiles penetrating soft clay at high velocities. There is excellent agreement between the measured and predicted velocity profiles using $\alpha = 0.27$, which is within the range deduced from the cyclic piezoball remoulding tests (0.19 to 0.33). In contrast, the best agreement that could be obtained without the inclusion of drag resistance required $\alpha = 0.38$, which is inconsistent with results from the cyclic piezoball remoulding tests and gave a much poorer match.

### 5.8 Conclusions

This paper describes a fully self-contained low cost MEMS-based IMU consisting of a tri-axis accelerometer and a three-component gyroscope, and considered sample data captured by the IMU during field tests on dynamically installed projectiles. Such data are important for understanding the soil-structure interactions that occur at the elevated shear strain rates associated with dynamic penetration events. To the authors’ knowledge these data are the first reported use of a 6DoF IMU for a geotechnical application.

A comprehensive framework for interpreting the IMU measurements so that they are coincident with a fixed inertial frame of reference was described and implemented to establish projectile rotations, accelerations and velocities during free-fall in water and embedment in soil. It is often the final embedment depth of a dynamically embedded
projectile that is of interest. The paper showed that for projectiles that tilt during free-fall, embedments calculated from the body frame acceleration measurements, rather than from accelerations transformed to an inertial frame of reference, led to derived embedment depths that were in error by up to 27%. In contrast, embedment depths derived from IMU data interpreted from within an inertial frame of reference were shown to be in excellent agreement with independent direct measurements.

The merit of collecting motion data during dynamic penetration events was demonstrated by using the IMU data to validate an embedment prediction model based on strain rate enhanced shear resistance and fluid mechanics drag resistance for dynamically installed anchors. In this demonstration the inclusion of drag resistance during embedment in soil was shown to be appropriate, as the measured and predicted velocity profiles were in excellent agreement. In contrast, when drag resistance was omitted an interface friction ratio inconsistent with the measured soil sensitivity was required to match the final embedment depth, and as a consequence the overall agreement between the measured and predicted profiles was much poorer.

In conclusion, the use of a reliable IMU with an appropriate interpretation framework is required to successfully apply these projectile-based geotechnical devices.
5.9 Figures

Figure 5.1 Schematic representation of the operational principle of: (a) MEMS accelerometers and (b) MEMS gyroscopes
Figure 5.2 Deep penetrating anchor

Figure 5.3 Dynamically embedded plate anchor

Figure 5.4 Instrumented free-falling sphere
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Figure 5.5 Inertial Measurement Unit

Figure 5.6 Measurements made in the body frame of reference
Figure 5.7 Body and inertial frames of reference

Figure 5.8 Location of test sites
Figure 5.9 Undrained shear strength profiles: (a) Firth of Clyde and (b) Lough Erne
Figure 5.10 (a) RV Aora and (b) Self-propelled barge
Figure 5.11 DEPLA field test procedure
Figure 5.12 Image capture from ROV camera showing the follower retrieval line at the seabed
Figure 5.13 Projectile rotations during free fall through water and soil penetration: (a) DEPLA, (b) IFFS, and (c) DPA
Figure 5.14 Projectile accelerations during free fall through water and soil penetration: (a) DEPLA, (b) IFFS, and (c) DPA
Figure 5.15 Projectile velocity profiles corresponding to free fall through water and soil penetration: (a) DEPLA, (b) IFFS, and (c) DPA
Figure 5.16 Comparision of IMU derived displacement measurements with those obtained using a draw wire sensor

Figure 5.17 DEPLA velocity profile derived from the IMU data measured at the Firth of Clyde test site and corresponding theoretical profiles
Chapter 6

A Release-to-Rest model for dynamically installed anchors

Chapter context: The paper presented in this chapter introduces a new Release-to-Rest model, which simulates the motion history of a dynamically installed anchor during freefall in water and dynamic embedment in soil. Data captured from dynamic installation tests with a 1:20 scale anchor instrumented with the IMU described in Chapter 5 in a lake in Northern Ireland (sub-objective 1), together with further tests reported by industry, are analysed. The analyses incorporate findings made in chapters 3 and 4, and the results used to validate the new R2R predictive tool (addressing sub-objective 3).

This chapter has been accepted for publication as:

Main objective: Develop calibrated and validated design tools for predicting the performance of dynamically installed anchors in soft soil

Sub-objective 1: Develop an experimental database, encompassing both centrifuge and field tests, for dynamically installed anchors as a means of establishing anchor performance in soft soil and for calibrating design tools.

Chapter 3: Soil response in the wake of dynamically installed projectiles.

Sub-objective 2: Answer the question as to how the soil responds in the wake of a dynamically installed anchor during dynamic embedment, establishing the conditions under which the hole formed by the passage of the anchor may close.

Chapter 4: Assessing the penetration resistance acting on a dynamically installed anchor in normally and over consolidated clay.

Sub-objective 3: Refine and calibrate analytical embedment models for dynamically installed anchors using motion data collected in the centrifuge and field tests from sub-objective 1.

Chapter 5: In situ measurement of the dynamic penetration of free-fall projectiles in soft soils using a low-cost inertial measurement unit.

Sub-objective 4: Establish design tools for predicting the monotonic capacity of dynamically installed anchors for a range of load inclinations, calibrated using field data from sub-objective 1.

Chapter 6: A release-to-rest model for dynamically installed anchors.

Completion of the above sub-objectives leads to the completion of the main objective and Chapter 8.

Chapter 7: Capacity of dynamically installed anchors as assessed through field testing and three dimensional large deformation finite element analyses.

Chapter 8: Conclusions derived from the work described in the thesis.
6.1 Abstract

Dynamically installed anchors are torpedo-shaped anchors that are installed by dropping them through the ocean such that they self-bury in the soft seabeds typically encountered in deep water. This paper presents and considers field data from reduced scale anchor tests at two sites to validate a new Release-to-Rest model for dynamically installed anchors. This model extends existing studies by considering the motion of the anchor from the point of release in the water column, modelling the drag resistance that acts on the anchor and its mooring line. Simulations from the model, together with data from the field tests, demonstrate the importance of considering the free-fall in water phase of installation, as the drag developed on the mooring line can be significant for larger release heights, reducing the anchor velocity as it arrives at the seabed by up to 44%. The soil embedment phase of the model also considers drag resistance on the anchor – and optionally on the mooring line – and accounts for the effect of the high anchor velocities on soil strength using a simple power law. Model simulations are shown in the paper to be in very good agreement with the anchor motion data measured in the field tests, resulting in predicted anchor embedments that are in agreement with the field database of over 100 anchor installations to an accuracy of ±10%.
6.2 Introduction

Dynamically installed anchors are large rocket or dart-shaped anchors that are installed by release from above the seafloor, such that they fall through the water column and self-bury within the seabed. The speed and ease of installation make them attractive options, particularly in deep water. Unlike driven piles and suction caissons that are directly installed to a predetermined depth in the seabed, dynamically installed anchors achieve a final embedment depth that is governed by the seabed strength and sensitivity, the velocity and acceleration of the anchor upon arrival at the seafloor, and the mass and geometry of the anchor. Geotechnical design of these anchors firstly requires that the final embedment depth is calculated as this is a prerequisite for designing the anchor for a given mooring line load. This calculation is complicated for a number of reasons. Firstly, the anchor velocity as it travels through the seabed is of the order of metres per second, leading to strain rates that are orders of magnitude higher than those associated with the measurement of soil strength. The implication of this is that the mobilised soil strength during dynamic penetration is higher than that measured in situ or in the laboratory, and this needs to be captured in the embedment calculations. Secondly, there is the potential for water to become entrained at the anchor-soil interface, which would reduce the nominal interface frictional resistance by an amount that is difficult to quantify. Finally, drag resistance acts on the anchor and the trailing mooring line during freefall in water and dynamic embedment in the seabed, which further complicates the analysis.

Experimental studies on anchor installation have typically been at laboratory scale, mainly from centrifuge testing in kaolin clay (e.g. O’Loughlin et al. 2004, 2009, 2013; Hossain 2014, 2015; O’Beirne et al. 2016) and consider only the response of the anchor during dynamic embedment in soil, ignoring the response of the anchor as it descends through the water column. By comparison, field studies are rare, although emerging (e.g. Zimmerman et al. 2009, Lieng et al. 2010; O’Loughlin and Blake, 2015; O’Loughlin et al. 2016). However, field studies often do not include the level of seabed characterisation or anchor instrumentation necessary to calibrate and validate prediction methods. This paper addresses this imbalance between laboratory and field experience by considering campaigns of anchor tests at two well characterised soft clay sites, using instrumented anchors with reduced scales of 1:20 and 1:3. The embedment phase of the tests is considered here, and the measurements made during installation are used to validate a new ‘release-to-rest’ (R2R) embedment model for dynamically installed...
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171 anchors. The R2R model extends anchor embedment models considered previously by simulating the entire anchor motion response starting from the point of release in the water column.

6.3 Release to rest anchor embedment model

O’Loughlin et al. (2013) showed that a first order approximation of the anchor embedment may be obtained by considering the total energy, $E_{\text{total}}$, of the anchor at the mudline – i.e. the sum of the kinetic energy at mudline and the remaining potential energy that would be dissipated during dynamic embedment in the seabed. A conservative estimation of anchor embedment, $z_{e,\text{tip}}$, can then be obtained from

$$
\frac{z_{e,\text{tip}}}{d_{\text{eff}}} \approx \left( \frac{E_{\text{total}}}{kd_{\text{eff}}^4} \right)^{1/3}
$$

where $d_{\text{eff}}$ is the effective anchor diameter (which accounts for the additional projected area of the anchor flukes) and $k$ is the gradient of undrained shear strength of the seabed. However, this empirical model does not account for variations in the interface friction along the anchor-soil interface, which (for a given soil strength) is expected to be higher in low sensitivity soils than in high sensitivity soils. This limitation was recognised by O’Loughlin et al. (2016), who adopted the concept of total energy, but presented the expected embedments as results from an anchor embedment model that explicitly considers the forces acting on the anchor and solves the Newtonian motion equation in a time-marching fashion. In this way the influence of soil sensitivity, strain rate dependence and anchor drag resistance can be accounted for. However, a remaining limitation is that the starting point in the analysis is the point of impact with the seabed, described in terms of the anchor velocity at the mudline. This is slightly unsatisfactory for two reasons. Firstly, it requires a pre-calculation or assumption regarding the response of the anchor as it descends through the water column, and secondly the initial starting point cannot be uniquely defined solely in terms of an impact velocity. The implications associated with this second point are demonstrated later in the paper. A more satisfactory approach is to consider the free-fall in water and embedment in soil consecutively, in what we refer to as the Release-to-Rest (R2R) model.

The R2R model defines the starting point at the release height in the water column, where the anchor’s velocity and acceleration are zero. Immediately upon release, the anchor acceleration tends towards the Earth’s gravitational acceleration, $g$, causing the anchor velocity to start to increase. As the anchor velocity increases, the anchor
acceleration decreases due to the fluid drag resistance that acts on the anchor and on the trailing mooring line. The anchor then arrives at the seabed with a calculated anchor acceleration and velocity, where the additional resistance due to the soil strength further decreases the anchor acceleration, eventually bringing the anchor to rest at some calculated depth in the seabed. The governing equation for the R2R model is

$$F_{net} = (m + m') \frac{d^2 z}{d t^2} = W_s - F_b - F_g - F_d$$  \hspace{1cm} (6.2)

where \(m\) is the anchor mass, \(z\) is anchor depth, \(t\) is time and \(W_s\) is the submerged anchor weight (in water). During free-fall in water the additional buoyancy due to the soil displaced by the anchor, \(F_b\), and the geotechnical resistance, \(F_g\), are zero and the only resistance acting on the anchor is the drag resistance, \(F_d\). For rigour, Equation 6.2 also includes the added mass, \(m'\), of water or soil that would be accelerated with the anchor. Whilst this is an important consideration for geometries with low aspect ratios (e.g. spheres, Morton et al. 2016), for long slender bodies such as the anchor considered here, \(m'\) is negligible and can be taken as zero (Beard 1981, Shelton et al. 2011). The drag resistance, \(F_d\), includes both pressure drag acting on the anchor and frictional drag acting on the mooring line, formulated as:

$$F_d = \frac{v^2}{2} \rho (C_{D,a} A_p + C_{D,l} A_l)$$  \hspace{1cm} (6.3)

where \(C_{D,a}\) and \(C_{D,l}\) are drag coefficients for the anchor and mooring line respectively, \(\rho\) is density (of water or soil), \(A_p\) is the anchor’s projected area and \(A_l\) is the mobilised contact area of the mooring line. As drag acts both in water and soil (Freeman and Schüttenhelm, 1990; Zhu and Randolph, 2011; Randolph and White, 2012; O’Loughlin et al. 2013; Sahdi et al. 2014; Morton et al. 2015; O’Beirne et al. 2016; Pang et al. 2016) the density, \(\rho\), evolves from that of water to soil in accordance with the anchor’s transition through the mudline.

During embedment in soil additional resistance from penetration into the seabed includes soil buoyancy, \(F_b\) calculated as the product of the soil density and the currently submerged anchor volume (in addition to potential added volume due to an open hole above the anchor; Aubeny & Shi 2006, Sabetamal et al. 2013; O’Loughlin et al. 2013, Chow et al. 2014, O’Beirne et al. 2015) and the geotechnical resistance \(F_g\), made up of conventional bearing and frictional resistance:
where $\alpha$ is an interface friction ratio (of limiting shear stress to undrained shear strength), $N_c$ is the bearing capacity factor for the anchor tip, padeye (if hole closure behind the anchor is assumed) or fluke and $s_u$ is the undrained shear strength averaged over the appropriate contact area – frictional area, $A_s$, or projected area, $A_p$. The term $R_f$ in Equation 6.4 accounts for the enhancement of soil strength due to the high anchor penetration velocity that invokes strain rates in the soil that are beyond that associated with undrained behaviour (Casagrande & Wilson 1951, Graham et al. 1983, Sheahan et al. 1996). As shown by Equation 6.4, these viscous strain rate effects are accounted for by employing a strain rate function, $R_f$, to scale the undrained shear strength according to the shear strain rate. $R_f$ is generally described using either power or semi-logarithmic functions to scale the soil strength as a ratio of the mobilised strain rate, $\dot{\gamma}$, to a reference strain rate, $\dot{\gamma}_{ref}$, associated with nominally undrained soil strength. The power law is generally preferred for problems involving higher orders of magnitude variation in strain rate (Biscontin and Pestana, 2001; Abelev and Valent 2009; O’Loughlin et al. 2013) and may be formulated as:

$$R_f = \left(\frac{\dot{\gamma}}{\dot{\gamma}_{ref}}\right)^\beta \approx \left(\frac{v/d}{(v/d)_{ref}}\right)^\beta$$

where the approximation of $\dot{\gamma} = v/d$ is used to avoid the complication of calculating the absolute magnitude of strain rate and its variation within the shear zone (O’Loughlin et al. 2013, Chow et al. 2014, O’Beirne et al. 2016).

The importance of modelling the free-fall in the water column phase of installation is demonstrated by Figure 6.3, which plots the dependence of anchor release height on the anchor net force and velocity at the mudline and the anchor final embedment depth. The results on Figure 6.3 were obtained for a full scale 80 t anchor, 13.2 m long (Lieng et al. 2010) in a seabed with an undrained shear strength profile given by $s_u = 1.5$ kPa/m (i.e with a zero mudline strength) and a soil sensitivity, $S_t = 2.5$. The parameters chosen for the embedment model are identical to those used later in the paper and a discussion on how the R2R model parameters are selected is provided later. The mooring line was modelled as shown in Figure 6.1b, such that a vertical section of mooring line extends from the rear of the anchor with a length that is initially three times the anchor length, but that increases by the current anchor displacement during free-fall in water. The soil
embedment calculations assume both taut and slack mooring lines during the soil embedment phase as these represent behavioural limits and indeed are the two cases considered in the field tests considered later in the paper. Figure 6.3 demonstrates the expected increase in anchor impact velocity with increasing release height, but limited to a release height of ~116 m, beyond which the impact velocity reduces. This is due to the additional resistance that develops on the anchor mooring line and that eventually causes the net anchor force to become negative. The implications for the final anchor embedment depth is that a higher anchor release height can lead to a lower anchor embedment, and that anchor embedment is not uniquely determined by impact velocity if drag resistance continues to develop on a non-slack mooring line during embedment in the seabed. The actual response of the mooring line when the anchor reaches the mudline is unclear and warrants further attention. However, in the simplifying scenario where the mooring line becomes slack when the anchor reaches the mudline, an anchor release height of 100 m produces an anchor impact velocity of 25.7 m/s and an anchor embedment depth of 29 m, whereas releasing the same anchor at the same site from 500 m would reduce the impact velocity to 18.2 m/s and the anchor embedment depth to 25.4 m.

6.4 Details of field tests

6.4.1 Description of sites

The field data considered in the paper include tests conducted by the authors on a 1:20 reduced scale anchor tested in a lake (Lough Erne) in Northern Ireland and tests reported by Sturm et al. (2011) on a 1:3 reduced scale anchor tested at the Troll Field in the North Sea. Water depths at the anchor testing locations were 3 to 19 m at Erne and 300 m at Troll. The lakebed at Erne is very soft normally consolidated clay with moisture contents in the range 270 to 520%, typically about 1.5 times the liquid limit, and a fines fraction of 95% (Colreavy et al. 2012). The very high moisture content is due to the very high proportion of diatoms that are evident from scanning electron microscopic images of the soil (Colreavy et al. 2012) and have an enormous capacity to hold water in the intraskeletal pore space (Tanaka & Locat, 1999). The seabed at Troll is well characterised and reported in detail by By and Skomedal (1992) and Lunne et al. (2006). Over the depth of interest for the anchor tests, the Troll seabed is very soft to firm, lightly overconsolidated clay, with moisture contents that are close to the liquid limit, in the range 47 to 70%, and a fines fraction of between 75 and 95%.
Figure 6.4 shows profiles of undrained shear strength, $s_u$, with depth, $z$, for the two sites. The Erne $s_u$ profiles were determined from piezoball and in-situ vane tests reported by Blake et al. (2015), whereas the Troll $s_u$ profiles were determined from piezocone tests calibrated against a combination of triaxial compression and extension tests and direct simple shear tests (By and Skomedal 1992). At Erne the strength profiles may be idealised as $s_u = 1.5z$ from $z = 0$ to 1.5 m, and by $s_u = 2.25 + 0.8z$ from $z = 1.5$ to 3 m, whereas at Troll the seabed strength is best idealised by $s_u = 5$ kPa from $z = 0$ to 2.5 m, and by $s_u = 5 + 2.69z$ from $z = 2.5$ to 15 m. The sensitivity of the Erne clay was in the narrow range $S_t = 1.9$ to 2.6 as assessed from piezoball cyclic remoulding tests (Colreavy et al. 2016), whereas the sensitivity of the Troll clay is $S_t = 5.5$ as determined from fall cone tests (Lunne et al. 2006).

### 6.4.2 Model anchors and instrumentation

The model anchors were based on an idealised design proposed by Lieng et al. (1999), featuring four clipped delta type flukes (separated by 90° in plan) with a forward swept trailing edge and an anchor shaft with an ellipsoidal tip. The anchors are shown in Figure 6.5.

The Erne 1:20 reduced scale anchor was fabricated from mild steel and has an overall length of 750 mm, a shaft diameter of 60 mm, a total projected area of $6.43 \times 10^{-3}$ m$^2$ and a dry weight of 203 N. The anchor was substantially solid with the exception of a watertight cylindrical void, 210 mm in length and 50 mm in diameter, at the upper end of the shaft that housed an inertial measurement unit (IMU) to measure accelerations and rotations. The anchor was deployed from drop heights in the range 0 to 19 m using either 4 or 12 mm diameter Dyneema SK75 rope. This rope is neutrally buoyant such that the mooring line remained vertical (and continued to the water surface) behind the advancing anchor.

The IMU used in the Erne anchor tests is a fully self-contained motion logger designed to capture the motion history of free-fall projectiles (Blake et al. 2016). The IMU includes a 16-bit three component micro-electro mechanical system (MEMS) rate gyroscope (ITG 3200) with a range of $\pm 2000$ °/s and a 13-bit three-axis MEMS accelerometer (ADXL 345) with a range of $\pm 16$ g. Data are logged by an mbed micro controller with an ARM processor to a 2 GB SD card at 400 Hz. Internal batteries are capable of powering the logger for up to 4 hours.
The MEMS accelerometer measures both linear and gravitational acceleration (depending on the anchor orientation) in three orthogonal body-frame axes that are common to both the IMU and the anchor. In order to distinguish the anchor’s linear acceleration component from the acceleration detected by the sensor — which will differ if the anchor pitches or rolls — acceleration measurements in the anchor’s ‘body’ frame were transformed to accelerations that are coincident with the Earth-fixed inertial frame using rotation matrices, described in detail by Blake et al. (2016). This is particularly important in the consideration of free-fall projectile data where pitch and/or roll angles are significant. However, in the Erne tests anchor tilt was consistently less than 2.5°, such that the misalignment between the body frame and the inertial frame of reference was negligible. Anchor velocity and displacement was obtained by numerically integrating the inertial frame ‘vertical’ acceleration, once to obtain velocity and twice to obtain displacement. This approach has been validated using independent displacement measurements (Blake et al. 2016), and was checked for each Erne test by sending a camera underwater to inspect markings on the anchor mooring line.

The Troll 1:3 scale anchor was also fabricated from mild steel, and had an overall length of 4.4 m, a shaft diameter of 0.4 m and a total projected area of 0.16 m². The shaft was partially solid, such that the total dry weight was approximately 28.5 kN. The anchor was released and retrieved using a mooring line that comprised 28 m of 36 mm stud link chain and 500 m of 38 mm diameter wire rope. The anchor instrumentation included a combination of triple axis ±3 g MEMS accelerometer logged at 200 Hz and a single axis ±20 g piezoelectric accelerometer logged at 20 Hz. Two anchors were considered in the tests, referred to here as ‘Anchor A’ and ‘Anchor B’. The instrumentation for Anchor A was located within the anchor shaft, whereas for Anchor B it was located alongside the anchor shaft in instrumentation pods, leading to a 19% and 4.5% increase in the projected and frictional areas respectively.

Anchor inclination was assessed before and after anchor motion from the triple axis accelerometers as the instrumentation did not include the rate gyroscopes necessary to determine inclination during anchor motion. This assessment resulted in maximum tilt angles of 5° for anchor release heights less than 50 m, and a maximum anchor tilt of 8° in one test where the anchor was released from 75 m. As these tilt angles are not sufficient to necessitate interpretation within a fixed inertial reference frame (O’Loughlin et al. 2014), the measured vertical axis acceleration was used to derive the anchor motion data, by numerical integration as described above for the Erne tests.
6.4.3 Experimental programme and test results

The Erne tests were carried out from either a fixed jetty or a floating pontoon (that was stabilised by two spud legs) in water depths up to 6.5 m, and from the deck of a barge in water depths of about 19 m. The anchor was initially suspended in the water at a preselected height above the seabed using a mobile crane for the jetty and pontoon tests, and a knuckle boom crane for the barge tests. Anchor release was initiated from above the waterline in Erne using a quick release swivel snap shackle. At Troll, the anchor tests were conducted using a survey vessel with a heave compensated crane. As was the case at Erne, the anchor was initially suspended in the water, but was released using a remotely operated vehicle (ROV) that swam to the release height to cut through a sacrificial sling (similarly to the configuration shown in Figure 6.1b).

A total of 103 anchor tests were conducted at Erne from release heights above the mudline predominantly in the range 0 to 5 m, but with a limited number from about 16 m. As described earlier, checks on the anchor release height and final anchor embedment depth were obtained using a camera that was lowered through the water to inspect markings on the mooring line. These checks were performed to ensure no gross error in the embedment determined from the IMU data, and typically gave embedment depths that were typically within 5% of those derived from the IMU measurements. Of the 103 Erne tests, 28 included IMU measurements and the remaining 75 relied on visual measurements of the anchor release height and anchor embedment depth. Eleven anchor tests were conducted at Troll, with release heights in the range 15.5 to 75 m. As at Erne, checks on the final anchor embedment depth were made using an ROV that swam to the mudline to inspect markings on the mooring line.

At Erne impact velocities were in the range 0 to 6.7 m/s, but were higher – in the range 12.8 to 15 m/s – at Troll due to the larger anchor scale. At both sites the anchor impact velocity was a function of the release height. However, release heights greater than that required to achieve the maximum velocity led to lower anchor impact velocities due to the additional drag resistance mobilised by the mooring line. This aspect is explored further later in the paper.

Tip embedments were in the range $z_{e,tip} = -1.1$ to $-1.9$ m at Erne compared with $z_{e,tip} = -7.4$ to $-8.7$ at Troll. This corresponds to a maximum of 2.5 anchor lengths at Erne and 2 anchor lengths at Troll, consistent with existing experience from centrifuge and field
tests (Medeiros 2002; O’Loughlin et al. 2004; Lieng et al. 2010). Although the higher strength gradient at Troll reduces the normalised embedment depth (i.e. $z_{e,\text{tip}}/L$), the higher soil sensitivity partly compensates for the higher strength gradient, such that the reduction is not as significant as might be expected (O’Loughlin et al. 2016).

### 6.5 Assessment of R2R model

#### 6.5.1 Model parameters

The parameters required for the R2R model include ‘conventional’ parameters that would be considered for deep penetration problems, e.g. driven piles and suction caissons, and other non-standard parameters, such as the drag coefficients $C_{d,a}$ and $C_{d,l}$ and the strain rate parameter, $\beta$. The conventional parameters include the capacity factor, $N_c$, that relates the bearing resistance on the anchor tip and flukes to the soil strength, and the interface friction ratio, $\alpha$, which is the ratio of the soil strength that is mobilised along the anchor flukes and shaft as frictional resistance. American Petroleum Institute guidelines recommend $N_c = 9$ for driven piles (API, 2002), although for the ellipsoidal tipped anchors considered here, static (nominally undrained) penetration tests suggest that $N_c = 12$ is more appropriate (O’Loughlin et al. 2009). Reverse end bearing at the anchor padeye will develop if the hole formed by the passage of the anchor closes during dynamic penetration. O’Beirne et al. (2015) reported experimental observations made using a high speed camera, which show that hole closure can occur at the same rate as dynamic embedment, but only if the padeye embedment depth exceeds a critical value for the dimensionless strength ratio, $s_u/\gamma'D$, at the anchor padeye. At Troll $s_u/\gamma'D$ at the anchor padeye was in the range 3.2 to 5.0, with achieved padeye embedment depths that are sufficient to develop full hole closure. Given the very low soil strength, full hole closure also appears a reasonable assumption for Erne (consistent with the approach adopted by Morton et al., 2016), although the very low effective unit weight would hamper a judgement based solely on $s_u/\gamma'D$.

The anchor flukes may be considered similar to the skirt tip of a caisson, which is typically modelled using $N_c = 7.5$, analogous to a deeply embedded strip foundation (Skempton 1951). The interface friction ratio, $\alpha$, is often taken as the inverse of soil sensitivity (Karlsrud et al. 1993; Anderson et al. 2005) and this approach has been shown to be appropriate for dynamically installed anchors when jacked-in under nominally undrained conditions (O’Loughlin et al. 2009). Recent studies have shown
that the strain rate parameter, β, appropriate for dynamic penetration problems is similar to that measured in variable rate penetrometer tests (O’Loughlin et al. 2013; Blake and O’Loughlin, 2015; Morton et al. 2016). This is because in both scenarios the strain rate effects are somewhat muted by rather extreme soil softening. Consequently, strain rate parameters from variable rate penetrometer tests tend to be at the lower end of the β = 0.05 to 0.17 (Jeong et al. 2009) quoted in the literature, e.g. β = 0.05 to 0.09 (Low et al. 2008, Chung et al. 2006), β = 0.06 to 0.08 (Lehane et al. 2009). Hence β = 0.05 to 0.09 is considered an appropriate range for dynamically installed anchors, provided that the strain rate function, Rf (Equation 6.5) is not applied over more than 3 orders of magnitude (O’Loughlin et al. 2013, Blake and O’Loughlin 2015, O’Beirne et al. 2016).

A mid-range β = 0.07 was taken here for both Erne and Troll. The use of Equation 6.5 also requires stipulation of the strain rate – approximated here by v/d – associated with the reference undrained shear strength. In the case of the tests considered here the reference undrained shear strength was determined from in situ penetrometer tests, using a 113 mm diameter piezoball at Erne and a 35.7 mm diameter piezocone at Troll, both penetrated at the industry standard v = 20 mm/s. This gives v/d_{ref} = 0.18 s\(^{-1}\) and 0.56 s\(^{-1}\) for Erne and Troll respectively. As noted by O’Loughlin et al. (2016), caution should be exercised if the reference undrained shear strength is that measured in laboratory element tests, as these involve strain rates that are typically five orders of magnitude lower than associated with a penetrometer test, whilst providing similar magnitudes of s_u (as shown by Figure 6.4b). This reflects the compensating effects of strain softening associated with the penetrometer strength and more moderate strain rate effects at strain rates associated with the laboratory test. Preference is given to in situ penetrometer strength measurements, as the extent of strain softening and the strain rates are closer to those associated with anchor installation.

Some studies of free-fall penetrometers and anchors in clay (e.g. Chow et al. 2014, Steiner et al. 2014, O’Loughlin et al. 2016) consider that strain rate effects are higher for frictional resistance than for bearing resistance, and modify Rf (Equation 6.5) when applying it to frictional resistance to obtain the best agreement with measurements. However, alternate studies (e.g. O’Loughlin et al. 2013, Blake and O’Loughlin 2015) consider that the measured response is best captured by applying the same strain rate enhancement (i.e. the same value of Rf) for frictional resistance as bearing resistance. As shown by O’Berine et al. (2016), strain rate dependency may be higher for frictional resistance, although only if a soil strength lower than the fully remoulded strength is
considered as the reference strength. This is consistent with the entrainment of a boundary layer of water at the body-soil interface during installation, as this would require a higher $R_f$ term when paired with $\alpha = 1/S_t$.

The drag coefficients for the anchor and line may be determined numerically for various anchor geometries and mooring line configurations (e.g. Øye 2000; Raie and Tassoulas 2009), although the free-fall in water phase of the tests considered here offers an opportunity for back-analysis of these parameters. This process is demonstrated in Figure 6.6 for tests involving anchor release heights of 1 to 16.5 m at Erne, and 16 to 75 m at Troll. The data show the expected increase in anchor velocity with travel distance through the water column, whilst also showing a subsequent reduction in anchor velocity in the tests involving the greatest release heights. As discussed earlier in the paper, this is due to the additional drag resistance acting on the mooring line, which causes the net anchor acceleration to become negative in the water column, slowing the anchor. Also shown in Figure 6.6 are R2R model predictions (for this water phase only). The best agreement between the model predictions and the test data was obtained using $C_{D,a} = 0.67$ or 0.7 and $C_{D,l} = 0.019$ or 0.008 for Erne and Troll respectively. The back analysed $C_{D,a}$ values are consistent with $C_{D,a} = 0.7$ determined in a similar manner for a geometrically different anchor at Erne (Blake and O’Loughlin, 2015) and $C_{D,a} = 0.63$ determined using computational fluid dynamics analyses Øye (2000). The frictional drag acting on the trailing mooring line can be considered analagous to the frictional shear stresses acting on an infinitely long cylinder. To the authors’ knowledge this problem has not been considered theoretically, possibly as most mooring lines mainly experience normal drag (i.e. due to fluid flow normal to the mooring line). However, the back analysed $C_{D,l} = 0.019$ at Erne is comparable to $C_{D,l} = 0.015$ as determined by Blake and O’Loughlin (2015). The rather different back analysed $C_{D,l} = 0.008$ for Troll may arise from how the wetted surface area, $A_l$, was considered for the complex chain geometry. This chain was modelled as an effective chain diameter, calculated by considering a unit cylindrical length with the same volume as the same unit of chain. This approximation, or indeed a potentially different flow regime around the more complex chain geometry (with a different surface roughness), may explain the lower back-analysed $C_{D,l}$ at Troll.
6.5.2 Model performance

Example comparisons of measured and predicted motion response of the anchor – selected to represent the range of anchor release heights at both sites – are provided in Figure 6.7 for Erne and Figure 6.8 for Troll. Input to the R2R model includes the seabed strength profile, the mooring line geometry and the anchor geometry and mass. The remaining input are the key R2R model parameters that were described in the previous section and are summarised in Table 6.2. The motion response is represented by both anchor velocity and the net resistance on the anchor (i.e. the product of the anchor mass and acceleration) for each of the Erne tests, but only for the 35 m release height at Troll as this is the only test for which acceleration data are presented in NGI (2008).

Prior to release, the anchor velocity, \( v = 0 \text{ m/s} \), and the net force is equal to the submerged anchor weight in water, \( F_{\text{net}} = 173 \text{ N} \) and \( 22.6 \text{ kN} \) for the Erne and Troll tests respectively. After release the anchor velocity increases, causing an increase in the fluid drag resistance and a corresponding decrease in \( F_{\text{net}} \). As the anchor transitions from water to soil, the velocity either increases or decreases depending on whether \( F_{\text{net}} \) is positive or negative. For the release heights of 0 to 3 m at Erne, \( F_{\text{net}} \) is positive as the anchor arrives at the lakebed, and remains positive – with a corresponding increase in anchor velocity – to \( z = -0.47 \text{ to } -0.64 \text{ m} \). This is most pronounced for the 0 m release height, in which the anchor was released from the lakebed such that the velocity at the soil surface was \( v = 0 \text{ m/s} \), but increased to a maximum \( v = 2.62 \text{ m/s} \) at \( z = -0.64 \text{ m} \). These observations simply relay that the submerged weight of the anchor in soil is greater than the combination of shear and drag resistance. For the 16.5 m release height at Erne (corresponding to 22 times the anchor length), \( F_{\text{net}} \) becomes negative during freefall in water after a free-fall distance of 8 m, beyond which the anchor velocity reduces. Consequently when the anchor arrives at the lakebed, the additional resistance due to soil strength causes an immediate further reduction in anchor velocity. A similar motion response was measured in the Troll tests, as shown by the experimental data on Figure 6.8a. The freefall phase of the tests and the transition to soil is captured accurately by the R2R model, including cases where the release height was sufficient to cause \( F_{\text{net}} \) to become negative and the anchor velocity to reduce. An exception to this is Test V0D9 at Erne, which was released from 2 m above the mudline (Figure 6.7b). Upon release \( F_{\text{net}} = 150 \text{ N} \), which is less than the submerged weight of the anchor in water \( (F_{\text{net}} = 173 \text{ N}) \), suggesting that the quick release mechanism only partly opened, imparting an upwards tensile force on the mooring line.
The soil penetration phase of the tests is also well described by the R2R model, albeit that the net penetration force in the Erne tests is generally over predicted for the final 0.25 to 0.5 m of penetration (0.33 to 0.67 anchor lengths). However, and more importantly, the final anchor embedment is predicted accurately for each test. This was achieved in the Erne tests by varying the interface friction ratio in the range $\alpha = 0.42$ to 0.47 to achieve the best match with the measurements, which is within the 0.38 to 0.52 range inferred from the piezoball cyclic remoulding tests. In the Troll tests the soil penetration phase and in turn the final anchor embedment depth was accurately predicted using $\alpha = 0.18$ as inferred from the inverse of the soil sensitivity. As discussed earlier in the paper, higher strain rate effects may be permitted for fictional resistance. This is generally modelled by including a parameter $n$ in Equation 6.5:

$$R_f = \left( n \frac{v/d}{(v/d)_{ref}} \right)^{\beta}$$  \hspace{1cm} 6.6

where $n = 1$ for bearing resistance, and a function of $\beta$ for frictional resistance

$$n = 2 \left( \frac{1}{\beta} - 1 \right)$$  \hspace{1cm} 6.7

Hence for $\beta = 0.07$, $R_f$ is approximately 26% higher for frictional resistance than for bearing resistance. Alternate predictions based on this approach are compared with the measurements and the existing predictions on Figure 6.9 for Test V12D1 at Erne (released from 3 m above the mudline) and Test 6 at Troll (released from 35 m above the mudline). The alternate predictions under-predict the final embedment depth by between 3 and 5%, which is to be expected as using Equation 6.6 with the parameters already established increases the frictional resistance. Agreement with the measurements requires a reduction in $\alpha$ by a factor equal to $n^\beta$, i.e. by the same amount that $R_f$ was effectively increased by. These revised predictions are exactly coincident with the original predictions obtained using Equation 6.5, but with the interface friction ratio reduced from $\alpha = 0.46$ to 0.37 for Test V12D1 at Erne and from $\alpha = 0.18$ to 0.14 for Test 6 at Troll. This reduction in $\alpha$ is consistent with water entrainment along the anchor-soil interface that would mobilise a lower (reference) soil strength than the remoulded soil strength, requiring a lower interface friction ratio. Similar observations have been noted in tests on steel catenary risers (Yuan et al. 2016), suction caissons (Gaudin et al. 2014) and in high-rate ring-shear tests (Tika and Hutchinson, 1999). Although the field data considered here do not confirm which approach – Equation 6.5
with \( \alpha = 1/S_\text{c} \) or Equation 6.6 with \( \alpha = 1/(n^0S_\text{c}) \) – best simulates the true behaviour, experimental data from both centrifuge and field studies on free-fall cone penetrometers (Chow et al. 2016; Steiner et al. 2014) are quite persuasive that Equation 6.6 should be used. This, coupled with the data considered here (and also in O’Beirne et al. 2016), infers that water entrainment does occur.

Drag resistance on the mooring line was modelled both during free-fall in water and penetration in soil for the Erne tests as the mooring line was buoyant, and consequently maintained in tension. At Troll, drag resistance on the mooring chain was only modelled during free-fall in water and assumed to cease when the anchor reached the seabed due to the expected change in momentum of the anchor relative to the mooring chain. Although this aspect of the problem is poorly understood, excluding mooring chain drag resistance during soil penetration led to predicted final anchor embedments that were within 2.5% of the measurements (for the tests considered in Figure 6.8) compared with up to 6.7% deviation for predictions that assumed the contrary. The differences are relatively small, but do support the assumptions made regarding the mooring chain behaviour for the Troll tests. Figure 6.7e and 6.8c also provide R2R predictions where mooring line drag resistance in water is ignored. In these high anchor release height instances, ignoring mooring line drag leads to an over estimation of anchor impact velocity by 35 to 44% and over estimation of anchor embedment by 20 to 24%.

The relative merit of the R2R model in predicting the entire database of Erne and Troll tests is shown in Figure 6.10. As the input to the model is a known anchor release height – rather than an assumed or measured impact velocity – Figure 6.10 is presented as anchor release height against anchor embedment depth. The predictions for Erne are provided for the two different mooring lines used in the tests (4 mm and 12 mm), whereas at Troll the predictions are provided for the two different model anchors used in the tests. The R2R model is seen to fit the Erne database well (Figure 6.10a) using \( \alpha = 0.5 \), giving predicted embedment depths that are within 10% of the measurements. Similar performance is observed on Figure 6.10b for the Troll tests, where \( \alpha = 0.18 \) leads to predicted anchor embedment depths that are within 10% of the measurements. This in turn corresponds to an uncertainty in anchor capacity that is also bound by 10%, but will be lower as the anchor submerged weight is included in the capacity.
6.6 Practical implications

In this section the R2R model is applied to some hypothetical design scenarios to demonstrate how the model may be readily applied in practice and also to explore the sensitivity of required anchor size to seabed strength profiles (intact and remoulded). In each design scenario the factored mooring design load at the anchor padeye is 5 MN. For simplicity we consider only the vertical component of the mooring line load and neglect the capacity contribution from the inverse catenary of the embedded mooring chain. However, these aspects may be readily included using analytical expressions that describe the interaction of vertical and horizontal loading (e.g. O’Beirne et al. 2015), and that give the chain load and its inclination at the anchor padeye (e.g. Neubecker and Randolph, 1995). Anchor capacity is calculated using the API approach for driven piles (O’Loughlin et al. 2004; Lieng et al. 2000, O’Beirne et al. 2015), but using bearing capacity factor \( N = 12, 9 \) and \( 7.5 \) for the anchor tip, padeye and flukes respectively (O’Loughlin et al. 2004). The interface friction ratio is taken as the inverse of the soil sensitivity for installation, but rising by a factor of two for capacity mobilisation, based on published experience with piles (Randolph, 2003) and suction caissons (Jeanjean, 2006). The anchor geometry is that considered throughout the paper (assuming the anchor shaft is 70% solid), but scaled in our spreadsheet implementation of the R2R model to meet the design anchor capacity. Similarly, the anchor release height can be readily optimised in the model to achieve the maximum velocity as the anchor impacts the mudline.

The first scenario considers a seabed with an undrained shear strength profile given by \( s_u = 1.5z \) and a soil sensitivity, \( S_t = 2 \), such that \( \alpha = 0.5 \). This first design scenario is satisfied by an anchor with an overall length, \( L = 14.25 \) m, a mass, \( m = 101 \) tonnes, released from 135 m above the mudline such that it embeds to \( z_{e,\text{tip}} = 37.6 \) m. In the second design scenario the soil sensitivity is increased to \( S_t = 5 \), such that \( \alpha = 0.2 \), which required an anchor that was 12\% longer and 39\% heavier (\( L = 15.9 \) m and \( m = 141 \) tonnes), released from 157 m with a final anchor embedment, \( z_{e,\text{tip}} = 50.9 \) m. In the third design scenario the initial soil sensitivity, \( S_t = 2 \), is retained, but the strength profile is increased to \( s_u = 3z \). This required an anchor with \( L = 12.5 \) m and \( m = 68 \) tonnes, 12\% shorter and 33\% lighter than that needed for the first scenario.

These simple design cases are informative as they reveal that whilst the anchor embedment depth will self-correct in line with variability or uncertainty in the seabed strength profile, the correction is not fully compensatory. The model simulations show
that the anchor performs better in stronger, less sensitive soils, reflecting the lower amount of energy dissipation during installation in the stronger seabeds.

6.7 Conclusions

Although dynamically installed anchors are an attractive and often a cost-effective anchoring solution, their take-up globally has been somewhat hampered by uncertainties on their striking the seabed within an acceptable spatial variation, and on achieving the targeted embedment depth in the seabed. The latter has been addressed in this paper through a new Release-to-Rest model for anchor installation. The model simulates the motion history of the anchor during free-fall in water and dynamic embedment in soil, providing as output the final anchor embedment depth that can then be used to calculate anchor capacity. The importance of considering the motion response in water is demonstrated through model simulations that highlight the role of drag resistance that develops on the trailing mooring line. The observations from these simulations are also reflected in measurements made in field tests that have been used in this paper to validate the model.

The database of field tests amounts to 114 anchor installations at two sites using 1:20 and 1:3 reduced scale anchors. Measurements made during the tests allowed the entire motion history of the anchor to be established and compared with simulations from the R2R model. The R2R model was seen to capture the motion response well, and was capable of predicting the entire database to within ±10% of the measurements, using model parameters that are typical of those considered for the installation of piles or suction caissons, in conjunction with possibly less-familiar parameters that describe the increase in penetration resistance with increasing anchor velocity. These “dynamic” parameters account for the increase in soil strength due to strain-rate effects and drag resistance that acts on the anchor and the mooring lines, and are shown here to be well constrained. The R2R model can be readily implemented – either on a spreadsheet or using a programming language – making it a useful tool for practitioners.
6.8 Tables

Table 6.1 Installation data for (a) Erne and (b) Troll tests

<table>
<thead>
<tr>
<th>Test reference</th>
<th>IMU data</th>
<th>Rope diameter (mm)</th>
<th>Release height (m)</th>
<th>Water depth (m)</th>
<th>IMU impact velocity (m/s)</th>
<th>Theoretical impact velocity (m/s)</th>
<th>IMU tip embedment (m)</th>
<th>Visually measured tip embedment (m)</th>
<th>Theoretical tip embedment for $a = 0.5$ (m)</th>
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188
*Anchor was prevented from embedding in the soil to compare IMU measurements with directly measured freefall distance

(a)

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<th>Test reference</th>
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<th>Release height (m)</th>
<th>Measured impact velocity (m/s)</th>
<th>Theoretical impact velocity (m/s)</th>
<th>Measured tip embedment (m)</th>
<th>Theoretical tip embedment for $\alpha = 0.182$ (m)</th>
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(b)
Table 6.2 R2R input parameters for Erne and Troll

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<td>Line drag coefficient, $C_{D,l}$</td>
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6.9 Figures

Figure 6.1 Dynamically installed anchors: (a) Deep Penetrating Anchor (Deep Sea Anchors 2009), (b) installation procedure (after Lieng et al. 2000)

Figure 6.2 Resistance forces considered in the R2R model: (a) free-fall in water and (b) dynamic embedment in soil
Figure 6.3 Dependence of anchor embedment on anchor release height
Figure 6.4 Undrained shear strength profiles for: (a) Erne and (b) Troll
Figure 6.5 Model anchors: (a) Erne (1:20 scale) and (b) Troll (1:3 scale)
Figure 6.6 Experimental and theoretical anchor velocity profiles in water: (a) Erne and (b) Troll
Figure 6.7 Experimental and R2R model net force and velocity profiles for Erne tests: (a) Test V12D1, $\alpha = 0.46$ (b) Test V0D9, $\alpha = 0.43$ (c) Test V2D6, $\alpha = 0.42$ (d) Test V3D1, $\alpha = 0.47$ (e) Test HD1, $\alpha = 0.43$
Figure 6.8 Experimental and R2R model profiles of anchor motion for Troll tests: (a) Test T6, (b) Test T8 and (c) Test T10, all modelled using $\alpha = 0.18$
Figure 6.9 Comparison of experimental and R2R model predictions for (a) Erne Test V12D1 and (b) Troll Test T6, with differing assumptions regarding strain rate effects and the mobilised interface friction ratio.
Figure 6.10 Performance of R2R model for the entire database of (a) Erne and (b) Troll tests
Chapter 7

Capacity of Dynamically Installed Anchors as assessed through field testing and three dimensional large deformation finite element analyses

**Chapter context:** The paper presented in this chapter investigates the capacity characteristics of dynamically installed anchors using pullout data retrieved following dynamic installations of the 1:20 scale model anchor described in Chapter 6 (from sub-objective 1). Experimental vertical capacities are compared with conventional predictive calculations for offshore driven piles in soft soil. Experimental capacities are also established over a range of load inclinations. Corresponding large deformation finite element analyses are used in unison with the field observations to establish a simple design tool for estimating the required size of a dynamically installed anchor for a given mooring design (addressing sub-objective 4).

This chapter has been published as:

Main objective: Develop calibrated and validated design tools for predicting the performance of dynamically installed anchors in soft soil

Sub-objective 1: Develop an experimental database, encompassing both centrifuge and field tests, for dynamically installed anchors as a means of establishing anchor performance in soft soil and for calibrating design tools.

Sub-objective 2: Answer the question as to how the soil responds in the wake of a dynamically installed anchor during dynamic embedment, establishing the conditions under which the hole formed by the passage of the anchor may close.

Sub-objective 3: Refine and calibrate analytical embedment models for dynamically installed anchors using motion data collected in the centrifuge and field tests from sub-objective 1.

Sub-objective 4: Establish design tools for predicting the monotonic capacity of dynamically installed anchors for a range of load inclinations, calibrated using field data from sub-objective 1.

Completion of the above sub-objectives leads to the completion of the main objective and Chapter 8.

Chapter 3: Soil response in the wake of dynamically installed projectiles.

Chapter 4: Assessing the penetration resistance acting on a dynamically installed anchor in normally and over consolidated clay.

Chapter 5: In situ measurement of the dynamic penetration of free-fall projectiles in soft soils using a low-cost inertial measurement unit.

Chapter 6: A release-to-rest model for dynamically installed anchors.

Chapter 7: Capacity of dynamically installed anchors as assessed through field testing and three dimensional large deformation finite element analyses.

Chapter 8: Conclusions derived from the work described in the thesis.
7.1 Abstract

The capacity of dynamically installed anchors in soft normally consolidated clay was examined experimentally through a series of field tests on a 1:20 reduced scale anchor. The anchors were installed through free-fall in water, achieving tip embedment of 1.5 to 2.6 times the anchor length, before being loaded under undrained conditions at various load inclinations. Vertical anchor capacities were between 2.4 and 4.1 times the anchor dry weight and were satisfactorily predicted using the API approach for driven piles. Anchor capacity under inclined loading increased as the load inclination approached horizontal; the field data indicated this increase to be up to 30% for the minimum achievable inclination of about 20 degrees to the horizontal. Corresponding large deformation finite element analyses showed a similar response, with the maximum capacity occurring at a load inclination between 30° and 45° to the horizontal. The finite element results demonstrate that, for the anchor geometry considered, an inclined load at the anchor padeye could be decomposed into ultimate vertical and moment loading at the anchor centroid. The establishment of a V-M yield envelope for the geometry investigated forms the basis of a simple design procedure presented in the paper.
7.2 Introduction

Dynamically installed anchors are rocket or torpedo shaped and are installed by allowing the anchor to freefall from a designated height above the seabed (see Figure 7.1). Since first introduced in the late 1990s (Lieng et al. 1999), a number of design variations have been proposed (Lieng et al. 1999; Medeiros 2001; Shelton 2007), with overall lengths in the range 12 to 15 m, and shaft diameters in the range 0.76 to 1.2 m. Hydrodynamic stability during freefall is enhanced through side fins (or flukes) positioned towards the upper end of the anchor and optional ballasting inside the tubular shaft. The freefall height is selected such that the anchor velocity at impact with the seabed is close to the anchor’s terminal velocity. The anchor embedment achieved after impact with the mudline depends mainly on the anchor mass and geometry, impact velocity and seabed strength profile. However the capacity of dynamically installed anchors is mainly determined by the soil strength at the final embedment depth. This will be higher in weaker soils and lower in stronger soils but with approximately the same strength (at the final average embedment depth) in either case.

The majority of performance data available in the public domain for dynamically installed anchors are based on reduced scale laboratory studies (O’Loughlin et al. 2004, 2013; Richardson et al. 2006, 2009; Audibert et al. 2006; Gaudin et al. 2013). These data suggest that anchor tip embedment depths of up to 3 times the anchor length are achievable in typical deep water clay deposits, and that vertical monotonic capacities are typically less than five times the dry weight of the anchor. Numerical studies such as by Raie and Tassoulas (2006) and de Sousa et al. (2011) report similar findings. Although results from field trials have been reported by Medeiros (2002), Brandão et al. (2006), Zimmerman et al. (2009) and Lieng et al. (2010), these data are generally concerned with the embedment depth achieved after freefall and do not focus on anchor capacity.

The lack of field data available for dynamically installed anchors prompted a series of field tests on a reduced scale dynamically installed anchor. Installation aspects of the tests are reported by O’Beirne et al. (2016b), whereas this paper considers anchor capacity following dynamic installation. Vertical monotonic capacity data are compared with predictions based on American Petroleum Institute (API 2002) guidelines for piles in tension, whereas monotonic capacity data under inclined loading are considered under a combined loading framework, where the interaction between horizontal, vertical and moment loading is based on results from large deformation finite element analyses.
Chapter 7

7.3 Site Details and Soil Properties

The anchor tests were carried out in Lower Lough Erne, a glacial lake located in Northern Ireland in water depths of 3 to 8 m. The lakebed is a very soft clay with high moisture contents in the range 270 to 520% (1.2 to 1.7 times the liquid limit) and high Atterberg limits with plastic limits of 130 to 180% and liquid limits of 250 to 315% (Colreavy et al. 2012). The unit weight of the soil is constant with depth and is marginally higher than that of water at 10.8 kN/m³. The very high moisture content and very low unit weight is considered to be due to the very high proportion of diatoms that are evident from scanning electron microscopic images of the soil (e.g. see Colreavy et al. 2014).

Colreavy et al. (2014) reported results from piezoball and in-situ shear vane tests conducted at the anchor test sites to depths of up to 11 m. The undrained shear strength \( (s_u) \) profiles over the depth of interest for the anchor tests are shown on Figure 7.2 and were derived from the net penetration resistance, \( q_{\text{net}}; s_u = q_{\text{net}}/N_{\text{ball}} \) where \( N_{\text{ball}} = 8.6 \) was selected through calibration with the in-situ shear vane data. The profiles can be conveniently approximated by an undrained shear strength gradient, \( k = 1.5 \text{ kPa/m} \) over the initial 1.5 m and \( k = 0.8 \text{ kPa/m} \) beyond 1.5 m. The sensitivity of the soil is in the narrow range \( S_t = 2 \text{ to } 2.5 \) as assessed from in-situ vane tests and piezoball cyclic remoulding tests (Colreavy et al. 2014).

7.4 Experimental details

7.4.1 Model Anchors

The 1:20 scale model anchor used in the field tests was based on an idealised design proposed by Lieng et al. (1999) and fabricated from mild steel. The overall anchor length was 750 mm and the shaft diameter was 60 mm. The anchor features four clipped delta type flukes (separated by 90° in plan) with a forward swept trailing edge as shown in Figure 7.3. The total anchor dry weight is approximately 203 N and the anchor dimensions are summarised in Table 7.1.

The anchor shaft is substantially solid with the exception of a watertight cylindrical void, 210 mm in length and 50 mm in diameter, at the upper end of the follower which housed an inertial measurement unit (for measuring accelerations and tilt during the embedment phase; see O’Beirne et al. 2016b). Although the anchor tip is
interchangeable (see Figure 7.3), only results using the ellipsoidal tip are considered here.

The anchor was deployed and retrieved using either 4 mm or 12 mm diameter Dyneema SK75 rope. This rope was selected because it was more flexible and much lighter than wire rope, whilst maintaining high tensile strength and stiffness (<0.1% extension at the operating loads for the anchor tests).

7.4.2 Experimental Procedure

The anchor tests were carried out from either a fixed vessel berthing jetty in water depths of 3 to 4.5 m or from a floating pontoon (stabilised by two spud legs) in water depths up to 8 m. Installation was achieved by releasing the anchor in water from a predetermined height above the lakebed. Anchor velocities at the mudline (impact velocity) varied with anchor drop height, and were typically up to 6.5 m/s for release heights of up to 4.5 m. This resulted in anchor tip embeddings in the range 1.125 to 1.95 m, or 1.5 to 2.6 times the anchor length. A minimum centre to centre spacing of 1.5 m (equivalent to 25 shaft diameters) was allowed between adjacent test sites to avoid interaction effects. Further details of the installation aspects of the tests are provided in O’Beirne et al. (2016b).

After installation a period of approximately 5 minutes elapsed before the anchor was extracted from the lakebed using a portable crane and a 2 t winch. This is considered as an immediate extraction with negligible change in soil strength after installation due to consolidation. For vertical loading the winch was positioned directly over the embedded anchor with the retrieval chain and loading hook extended to allow for approximately 1.5 to 2 m of vertical displacement (see Figure 7.4a). For inclined loading, the retrieval line was connected to the winch via a pulley located at the base of the crane and the required inclination was achieved by positioning the crane at a specified horizontal distance from the drop site (see Figure 7.4b). This methodology assumes that the angle subtended by a fixed line from the padeye at the top of the anchor shaft to the base of the pulley is the same as the angle subtended by the anchor retrieval line at the padeye when the anchor reaches its peak capacity (i.e. the rope cuts through the soil to adopt an almost straight line). Calculations using the Neubecker and Randolph (1995) chain solution with the data measured in the inclined loading tests confirmed that the difference between the line angle at the mudline and the line angle at the padeye was less than 3°.
The extraction load was measured using an S-shaped 500 kg load cell connected in series between the loading hook and the anchor retrieval line. Displacement of the retrieval line was measured using a 5 m draw-wire sensor (string potentiometer) connected in parallel with the retrieval line (see Figure 7.4). The 24-Bit Data Acquisition System, instrument amplifiers and power supply were housed within a ruggedised container (Figure 7.4).

The displacement rate of the anchor retrieval line during extraction was 6.5 to 7.5 mm/s. As capacity data measured during extraction of the anchor are assessed relative to the undrained shear strength, it is important that the strain rates in the soil during extraction are comparable to those associated with the test used to quantify the soil strength. In this instance the in situ strength was measured using a 113 mm piezoball penetrated at the standard rate of 20 mm/s, which is expected to give undrained conditions. The average strain rate may therefore be approximated by \( v/D_{\text{ball}} = 0.18 \text{ s}^{-1} \), which is similar to that associated with extraction of the anchor; \( v/D = 0.11 \) to 0.13 s\(^{-1}\).

### 7.5 Test results

The database of 89 field tests is summarised in Table 7.3 and Table 7.4 for vertical and inclined loading respectively. In the vertical loading tests, the anchor release height and accordingly the anchor embedment depth, varied in an attempt to understand the dependence of embedment depth on vertical monotonic capacity. The embedment depths in these tests were in the range \( z_{e,\text{tip}} = 1.12 \) to 1.95 m, equivalent to 1.5 to 2.6 times the anchor length, achieved from an anchor release height in the range 0 (i.e. released with the anchor tip at the mudline) to 5.06 m. This is in good agreement with 1.9 to 2.4 times the anchor length from field tests on a geometrically identical 79 t anchor with \( L = 13 \) m and \( D = 1.2 \) m (Lieng et al. 2010) and other centrifuge and field experience as detailed in Table 7.2. In the inclined loading tests, the anchor release height was maintained at a constant 2.5 m, in an attempt to achieve approximately the same embedment depth in each test \( (z_{e,\text{tip}} \sim 1.7 \) m, 2.27L) and facilitate comparison of capacity in the analysis.

#### 7.5.1 Vertical Monotonic Loading

Typical load displacement responses measured during vertical monotonic loading are provided on Figure 7.5. The examples provided on Figure 7.5 represent the vertical capacities associated with the range of embedment depths achieved in the tests. As is to
be expected, higher vertical capacities correspond with deeper embedment depths where higher undrained shear strength was mobilised.

It is clear from Figure 7.5 that considerable mooring line displacement (and hence vertical anchor displacement) is required before the ultimate vertical capacity \( V_{ult} \) is mobilised. This displacement is in the range \( \Delta = 225 \) to 310 mm (3.8D to 5.1D) and increases with increasing capacity. This range is much higher than \( \Delta = 0.7D \) to 1.1D reported by Richardson (2008) from centrifuge tests on a geometrically similar dynamically installed model anchor embedded at 1.4L in normally consolidated kaolin clay (also shown on Figure 7.5), and also higher than \( \Delta = 2.7D \) reported by de Sousa et al. (2011) from finite element analyses of a vertically loaded dynamically installed anchor with an initial embedment depth of 1.93L. Although the stiffness of the lakebed soil was not measured, Figure 7.5 confirms that the load-displacement response in the field tests is much softer than in the centrifuge tests, with the result that a much larger displacement was required to mobilise capacity.

The centrifuge data included on Figure 7.5 correspond to prototype reconsolidation periods (after dynamic installation) in the range 27 days to 68.4 years. The tests for the shorter reconsolidation periods (27 and 200 prototype days) are characterised by a sharp increase in load towards an initial peak capacity, followed by a rapid drop in load and a subsequent increase towards a secondary peak capacity of lower magnitude than the initial peak. This response, which has also been observed for suction caissons (Jeanjean et al. 2006), is considered to be due to frictional and bearing resistances reaching their respective peak values at different times (Richardson et al. 2009). Although the centrifuge data indicate that this response becomes more prominent as the reconsolidation period reduces, the field tests did not exhibit this response despite the non-dimensional reconsolidation time, \( T = c_t t/D^2 \), which is approximately one order of magnitude lower than in the centrifuge tests with the shortest reconsolidation period. The reason for this disparity is not wholly understood, although it is quite possibly due to the lower apparent stiffness of the lakebed soil.

Figure 7.6 demonstrates the dependence of the ultimate vertical anchor capacity on the anchor tip embedment for all 61 anchor tests involving vertical monotonic loading. Ultimate capacities are in the range \( V_{ult} = 490 \) to 820 N, increasing with anchor tip embedment, and are equivalent to 2.4 to 4 times the anchor dry weight, W. On the whole, this range is highly comparable with centrifuge and field experience, as
summarised in Table 7.2. Higher capacities, up to 8.1Wš, have been derived from finite element analyses and reported by Lieng et al. (2000). However this is considered to be an artefact of the wished in place anchor embedment depth, which is approximately 60% higher than expected given the geometry and weight of the anchor and the relatively high seabed strength (s₀ = 6 + 1.63z).

Also shown on Figure 7.6 is the theoretical vertical anchor capacity calculated using the API framework for offshore driven piles in clay:

\[ V_{\text{ult}} = W_s + F_{\text{bear}} + F_{\text{frict}} \]  

where \( W_s \) is the submerged weight of the anchor in soil, \( F_{\text{bear}} \) is the end bearing resistance and \( F_{\text{frict}} \) is the shaft frictional resistance expressed in the form:

\[ F_{\text{frict}} = \alpha s_u A; F_{\text{bear}} = N_c s_u A \]

where \( \alpha \) is an adhesion factor (of limiting shear stress to undrained shear strength), \( N_c \) is the bearing capacity factor (applied to the anchor tip, padeye and top/bottom of the flukes), \( s_u \) is the undrained shear strength averaged over the relevant contact area \( A \) (i.e. averaged over the shaft and wall area for \( F_{\text{frict}} \) and averaged over the projected area for \( F_{\text{bear}} \)). The predicted vertical anchor capacity shown on Figure 7.6 was obtained by modelling the anchor flukes as deeply embedded strip footings using \( N_c = 7.5 \) (Skempton 1951) while \( N_c = 12 \) was adopted for the ellipsoidal tip (O’Loughlin et al. 2009) and \( N_c = 9 \) was adopted for the padeye (given the similarity of the padeye with a deeply buried circular footing; Skempton 1951). The best fit between the measurements and the predictions was obtained using \( \alpha = 0.5 \), which is within the range \( \alpha = 1/S_t = 0.4 \) to 0.5 and is to be expected given that negligible reconsolidation would have occurred between installation and extraction. As shown by Richardson et al. (2009), anchor capacity is expected to increase significantly following post-installation consolidation.

### 7.5.2 Inclined Monotonic Loading

Typical load displacement responses measured during inclined loading are provided on Figure 7.7, together with a reference case for pure vertical loading test (V8D5, \( z_{e,\text{tip}} = 2.15L \) from Figure 7.5). The initial embedment depth for each inclined loading test was \( z_{e,\text{tip}} \sim 1.7m (\approx 2.27L) \) achieved after release from a constant drop height of 2.5 m. Figure 7.7 indicates that inclined anchor capacity, \( F_i \), increases as the loading angle reduces (i.e. approaches horizontal loading), increasing by 38% as the loading angle
reduces from $\theta = 90^\circ$ to $33^\circ$. This is to be expected as mobilisation of bearing resistance from the anchor flukes will increase as the load inclination reduces.

Figure 7.7 also shows that the displacement required to mobilise maximum anchor capacity increases with reducing load inclination. Again, this is to be expected as the mobilisation displacement will be a ratio of the relevant anchor dimension during loading, which will be higher for inclined loading than for vertical loading owing to the projection of the anchor flukes into the direction of loading. The mobilisation displacement is approximately 2.6 times higher for a load inclination of $33^\circ$ than for vertical loading ($\theta = 90^\circ$). Similar observations from numerical studies have been reported by Lieng et al. (2000) and de Sousa et al. (2011), although in these cases the mobilisation distance is difficult to determine as the small strain finite element analyses did not reach a peak capacity.

7.5.2.1 Three-dimensional large deformation finite element analyses

As the padeye is located at the top of the anchor, inclined loading will comprise vertical, horizontal and moment components. The interaction of these components is complex, particularly for irregular geometries such as the dynamically installed anchor considered here. In view of this, a three-dimensional (3D) finite element study was carried out to explore the interaction between vertical, horizontal and moment loading. Previous finite element studies of dynamically installed anchors using conventional small strain finite element methods demonstrate ever-increasing load magnitudes where the calculation was forced to stop due to severe entanglement of soil elements. The ‘ultimate’ capacity was then defined through some mandatory criteria such as the maximum capacity achieved prior to such severe entanglement (de Sousa et al. 2011; Lieng et al. 2000). A large deformation finite element (LDFE) approach termed ‘remeshing and interpolation technique with small strain (RITSS)’ proposed by Hu and Randolph (1998) was instead used for this study. RITSS was developed to overcome mesh distortion by periodic mesh regeneration of the deformed soil geometry. Stresses and soil properties are mapped from the old to the new mesh, where small strain calculations can then be carried out. Abaqus/Standard was called by a main program coded in Fortran to generate the mesh and to complete updated Lagrangian calculations automatically, which is facilitated by programs coded beforehand in Python (the script language of Abaqus). A detailed procedure and verification of the 3D Abaqus-based RITSS can be found in Wang et al. (2010; 2011) and Wang and O’Loughlin (2014).
The dynamic penetration of the anchor was not considered in the LDFE analyses, i.e. the anchors were wished in place at a tip embedment, \( z_{\text{e,tip}} = 1.7 \) m, typical of that achieved in the field tests involving inclined loading. The anchor was considered rigid and the soil was bonded to the anchor (i.e. anchor-soil interface was fully rough) in order to reduce computational cost. The soil was regarded as an elastic-perfectly plastic material with Tresca yield criterion, a Young’s modulus of \( 500s_u \) and Poisson’s ratio of 0.49 to approximate constant volume under undrained conditions. Although Figure 7.5 indicates that a lower Young’s modulus would be more appropriate for this soil, adjustment of the Young’s modulus will not affect the ultimate anchor capacity (Wang et al. 2010, Chen et al. 2013) and would unnecessarily prolong the computational duration of each analysis. The undrained shear strength was modelled in accordance with Figure 7.2, using a mudline strength of zero and a strength gradient, \( k = 1.5\) kPa/m to a depth of 1.5 m and \( k = 0.8\) kPa/m thereafter. An inclined displacement, rather than an inclined force, was applied to the anchor padeye to obtain apparent ultimate capacity.

Second-order tetrahedral elements were used to mesh the soil and anchor, with typical element sizes around the anchor of 0.09D. Although the ultimate capacities would be reduced marginally had a finer mesh been employed, the computational effort would have been unacceptably high. The geometrical symmetry of the anchor meant that only half the anchor and surrounding soil needed to be considered in the simulations. The total number of elements used in the analyses ranged between 200,000 and 220,000.

The load-displacement response predicted by small strain FE and LDFE approaches are shown in Figure 7.8 for load inclinations of 0, 45 and 90°. The small strain calculations stopped at a normalised displacement in the range \( \Delta/D = 0.4 \) to 0.6 due to numerical non-convergence caused by element entanglement. No ultimate capacities were achieved, which highlights the limitation of the small strain FE approach. The LDFE results follow the small strain results to about \( \Delta/D = 0.2 \), and the anchor load at \( \Delta/D > 0.6 \) remains nearly constant with anchor displacement.

The LDFE result for pure vertical loading is \( V_{\text{ult}} = 1204 \) N, which is higher than the API predictions (and hence the average experimental result, see Figure 7.6) for \( z_{\text{e,tip}} = 1.7 \) m as the LDFE analyses assumed a fully rough interface, inferring \( \alpha = 1 \), compared with \( \alpha = 0.5 \) used in the API predictions. Attempts were made to model reduced friction at the anchor-soil interface by specifying frictional contact on the interface. However, the updated Lagrangian calculations (within the finite strain theory) considering finite sliding along the frictional interface became non-convergent or unstable, due to the
complex 3D geometry of the anchor with sharp geometrical changes between the shaft and fluke surfaces and extremely small element sizes around the anchor that posed problems in determining the contact state at the soil-anchor interface.

In order to benchmark the LDFE result for vertical loading, additional API predictions were obtained using $\alpha = 1$. These gave $V_{\text{ult}} = 1034$ N, which is 14% lower than the LDFE prediction. This difference can be attributed to contact assumptions and mesh density. As the soil in the LDFE analyses is bonded to the anchor interface, the shear surface is along the integration points that are located within the soil elements adjacent to the anchor interface. This has the effect of a larger contact area mobilised during shearing.

Figure 7.8 shows that anchor capacity increases by approximately 9% as $\theta$ changes from 90° (vertical loading) to 0° (horizontal loading), and by approximately 18% as $\theta$ changes from 90° to 45°. This is made clearer by Figure 7.9 which compares anchor capacity for different load inclinations. Also shown on Figure 7.9 are small strain finite element results reported by de Sousa et al. (2011) for a geometrically dissimilar dynamically installed anchor, and the field data considered in this paper. Anchor capacity data are normalised by the anchor’s dry weight to permit comparison. The numerical capacities are seen to increase from $\theta = 0^\circ$ to a maximum value at $\theta = 30^\circ$ (LDFE) and $\theta = 45^\circ$ (small strain FE, de Sousa et al., 2011), before reducing to a minimum value at $\theta = 90^\circ$. The experimental data appear to follow this trend, albeit that comparison is not possible for $\theta < 21^\circ$, due to the minimum load inclinations achievable in the field tests. As discussed previously, the LDFE predictions are expected to be higher than the experimental data for vertical loading ($\theta = 90^\circ$) as the LDFE analyses assumed a fully rough interface whereas back analysis of the experimental data suggest $\alpha = 0.5$ (see Figure 7.6). As the load inclination reduces, more bearing resistance is mobilised in the failure mechanism. As bearing resistance is known to be a function of the interface friction ratio (e.g. Randolph and Houlbsy 1984; Martin and Randolph 2006), at low values of $\theta$ the LDFE predictions (using $\alpha = 1$) are sensibly higher than the experimental data, for which the mobilised interface friction ratio is expected to be closer to the inverse of the sensitivity (i.e. $\alpha = 0.4$ to 0.5).

An indication of the transition in failure mechanism as the load inclination changes is provided in Figure 7.10. For pure vertical movement the soil displacement vectors are also vertical, highlighting the frictional mode of resistance. As the load inclination
reduces, the soil displacement vectors become increasingly normal to the anchor surface as more bearing resistance is mobilised in the failure mechanism.

For horizontal loading at the padeye ($\theta = 0^\circ$), the mechanism is broadly similar to that observed when the anchor was subjected to pure rotation about its centroid (or load reference point, see Figure 7.11). This is an important observation, which suggests that for a horizontal load applied at the anchor padeye, the anchor does not experience any horizontal loading component at the load reference point, but only a moment component. Indeed, the maximum rotational capacity derived from horizontal loading at the padeye ($M_{\text{max}} = 385 \text{ Nm}$) is in good agreement with analyses in which the anchor was subjected to pure rotation about the load reference point, representing the ultimate moment capacity $M_{\text{ult}}$ (at $V = H = 0$) = 393 Nm. The relative difference in $M_{\text{max}}$ and $M_{\text{ult}}$ is 2%, which is sufficiently low that inclined loading at the anchor padeye can be decomposed solely into a vertical and moment component at the load reference point (as shown by Figure 7.11). Additional analyses with $k = 1$ kPa/m and $k = 3$ kPa/m resulted in respective $M_{\text{max}}$ values that were 3.7% and 0.3% lower than $M_{\text{ult}}$, confirming that this approach is reasonable over the range of strength profiles of relevance to dynamically installed anchors. It is worth noting that this approach will not be appropriate for other types of dynamically installed anchors (e.g. Shelton 2007), where the anchor padeye is eccentrically located from the anchor shaft (Wei et al. 2015).

Another important outcome from the LDPE analyses is the constant ratio between the ultimate vertical capacity $V_{\text{ult}}$ (at $H = M = 0$) and the maximum horizontal capacity $H_{\text{max}}$; $H_{\text{max}}/V_{\text{ult}} = 1.1$ for the particular anchor geometry considered here. This result can be used to establish a simplified design method, as described later.

### 7.5.2.2 Normalized Horizontal and Vertical Load Components

The capacity of foundations under combined vertical, horizontal and moment loading is commonly assessed by yield envelopes that describe the interaction between the three loading components. The literature includes numerous examples of such yield envelopes for shallow foundations (Bransby and O’Neill 1999, Gourvenec and Randolph 2003, among others), and a few for embedded anchors (Cassidy et al. 2012, de Sousa et al. 2011). The dynamically installed anchor investigated here represents a particular case where the location of the padeye results in a combination of vertical and moment loading applied at the anchor centroid, as demonstrated in the previous section.
The closed form expression commonly used to fit yield envelopes proposed by Bransby and O’Neill (1999) can therefore be reduced to the two components $V$ and $M$ as follows:

$$f = \left( \frac{V}{V_{ult}} \right)^q - 1 + \left( \frac{M}{M_{ult}} \right)^{m/p} = 0$$  \hspace{1cm} (7.4)$$

where the best fit with the LDFE data is obtained using $q = 4.49$, $m = 0.19$, and $p = 0.1$ (see Figure 7.12). Also shown on Figure 7.12 are the experimental data, for which the moment $M$ was simply calculated as the horizontal component of the load multiplied by the lever of arm (i.e. the distance between the padeye and the anchor centroid, $e_z$) and $M_{ult} = 255 \text{ Nm}$, giving $M_{ult}/V_{ult} = 0.36 \text{ m}$, broadly consistent with $M_{ult}/V_{ult} = 0.32 \text{ m}$ from the LDFE analyses.

### 7.5.2.3 Design Procedure

The following design procedure may be used to provide a first order estimation of the anchor scale required for a given mooring design. As anchor capacity depends on embedment depth and overall scale, the procedure requires an estimate of the anchor embedment depth, which for the first order approximation considered here, may be determined by considering the total energy of the anchor at the mudline (O’Loughlin et al. 2013):

$$\frac{z_{e,tip}}{D_{eff}} \geq \left( \frac{\frac{1}{2}mv_i^2 + m'g z_{e,tip}}{kD_{eff}^4} \right)$$  \hspace{1cm} (7.5)$$

where $m$ is the anchor dry mass, $v_i$ is the anchor impact velocity (assumed or estimated from hydrodynamic drag theory), $m'g$ is the submerged weight of the anchor in soil and $D_{eff}$ is the effective anchor diameter (accounting for the additional projected area of the anchor flukes).

The procedure also requires a means of calculating the load inclination at the anchor padeye. This may be obtained by numerically integrating the differential equations for a section of embedded chain, or estimated by solving the Neubecker and Randolph (1995) chain solution:

$$e^{\mu(\theta - \theta_m)}(\cos \theta_m + \mu \sin \theta) - \cos \theta - \mu \sin \theta = E_n d N_{c,chain} \left( s_{um} z_{e,pad} + \frac{kz_{e,pad}^2}{2} \right) \left( 1 + \frac{\mu^2}{F_i} \right)$$  \hspace{1cm} (7.6)$$
where $\theta_m$ is the mooring line angle at the mudline, $E_n$ is a multiplier giving the effective chain width in the direction normal to the chain, $d$ is the nominal chain diameter, $N_{c,\text{chain}}$ is the bearing capacity factor for the chain, $s_{\text{um}}$ is the undrained shear strength at the mudline, $z_{e,\text{pad}}$ is the depth to the anchor pad eye and $\mu$ is the chain-soil friction coefficient.

The design procedure includes the following six steps:

1. Resolve the factored design load at the anchor pad eye, $F_i$, into horizontal and vertical components, $V = F_i \sin \theta$ and $H = F_i \cos \theta$, where $\theta$ may be estimated from Equation 7.6.
2. Recalling that $H_{\text{max}} = 1.1 V_{\text{ult}}$ from the LDFE analysis (for the anchor investigated), Equation 7.4 can be rewritten as
   \[
   f = \left( \frac{F_i \sin \theta}{V_{\text{ult}}} \right)^{4.49} - 1 + \left( \frac{F_i \cos \theta}{1.1 V_{\text{ult}}} \right)^{0.19} = 0
   \]
   which can be solved to find $V_{\text{ult}}$.
3. Assume an overall anchor scale to give the anchor mass, $m$, and geometry.
4. Given the design $s_u$ profile, solve Equation 7.5 to calculate the anchor tip embedment depth, $z_{e,\text{tip}}$.
5. Calculate the axial capacity using the API (2000) approach for driven piles (Equation 7.1).
6. Repeat steps 2 to 5 until the axial capacity is at least equal to $V_{\text{ult}}$.
7. Check $\theta$ for the calculated anchor embedment using Equation 7.6 and repeat from step 1 if $\theta$ differs from that previously determined.

An example application is now considered, making use of the design procedure outlined above. In this example the undrained shear strength profile is described by $s_{\text{um}} = 0$ kPa and $k = 1.5$ kPa/m, $N_c = 12, 9$ and $7.5$ for the anchor tip, pad eye and flukes respectively, $\alpha = 1$ and the factored mooring design load (at the pad eye), $F_i = 5$ MN. Other relevant input data are summarised in Table 7.3. This results in a load inclination at the pad eye, $\theta = 44^\circ$. With $F_i = 5$ MN this gives $V_{\text{ult}} = 4.3$ MN ($V/V_{\text{ult}} = 0.81$), which could be satisfied using an anchor with an overall length of 12.9 m and a dry mass of 78.1 tonne.

Although the procedure is strictly only valid for the anchor geometry considered in this paper (and shown in Figure 7.3), a similar approach may be developed for other geometries.
7.6 Conclusions

This paper considers reduced scale field data to assess the capacity of dynamically installed anchors under both vertical and inclined monotonic loading. Vertical anchor capacities were in the range 2.4 to 4.1 times the anchor dry weight (reflecting the range in anchor embedment depth), comparable with previous findings, and can be satisfactorily predicted using the API approach for driven piles. Anchor capacity under inclined loading increased as the load inclination approached horizontal; the field data indicated this increase to be up to 30% for the minimum achievable inclination of about 20 degrees to the horizontal.

The interaction between vertical and horizontal (and hence moment) capacities was explored using large deformation finite element analyses. Results from the numerical analyses were consistent with the experimental data, although actual capacities were in all cases overestimated due to the different interface roughness conditions in the field tests and in the numerical analyses. The maximum anchor capacity was at a load inclination of between 30° and 45° to the horizontal, and is approximately 19% higher than for pure vertical loading.

The LDFE analyses showed that for the anchor geometry considered here, where the anchor padeye is at the upper end of anchor’s central axis, an inclined load can be decomposed solely into vertical and moment loading at the centroid of the anchor. The resulting V-M load interaction, was shown to be in good agreement with the experimental data, and formed the basis for a proposed design procedure for scaling a dynamically installed anchor under inclined loading. Although the procedure is strictly only valid for the anchor geometry considered in this paper, a similar approach may be developed for other geometries, based on the establishment of V-M yield envelope through large deformation finite element analyses.
### 7.7 Tables

**Table 7.1 Anchor dimensions**

<table>
<thead>
<tr>
<th>Anchor Length (Excluding Padeye)</th>
<th>Symbol</th>
<th>Dimension (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anchor Diameter</td>
<td>L</td>
<td>750</td>
</tr>
<tr>
<td>Tip Length</td>
<td>D</td>
<td>60</td>
</tr>
<tr>
<td>Shaft Length</td>
<td>L_{tip}</td>
<td>114</td>
</tr>
<tr>
<td>Fluke length - segment 1</td>
<td>L_{fluke1}</td>
<td>100</td>
</tr>
<tr>
<td>Fluke length - segment 2</td>
<td>L_{fluke2}</td>
<td>240</td>
</tr>
<tr>
<td>Fluke length - segment 3</td>
<td>L_{fluke3}</td>
<td>30</td>
</tr>
<tr>
<td>Total Fluke Length Along Shaft</td>
<td>L_{fluke}</td>
<td>370</td>
</tr>
<tr>
<td>Fluke Width</td>
<td>w_{fluke}</td>
<td>90</td>
</tr>
<tr>
<td>Fluke Thickness</td>
<td>t_{fluke}</td>
<td>10</td>
</tr>
</tbody>
</table>

**Table 7.2 Comparison of vertical monotonic capacity with previously reported data from centrifuge, field and numerical studies**

<table>
<thead>
<tr>
<th>Study</th>
<th>Methodology</th>
<th>Anchor</th>
<th>W_d (kN)</th>
<th>V_{u0}/W_d</th>
<th>z_{u0}/L</th>
<th>s_e (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lieng et al. (2000)</td>
<td>Numerical</td>
<td>L = 13.25 m D = 1.2 m 4 wide flukes L = 15 m</td>
<td>740</td>
<td>4.7 - 8.1</td>
<td>2.5</td>
<td>6+1.63z</td>
</tr>
<tr>
<td>Medeiros (2002)</td>
<td>Field</td>
<td>D = 1.07 m 4 narrow flukes</td>
<td>950</td>
<td>3.1 - 3.5</td>
<td>2.4</td>
<td>5+2z</td>
</tr>
<tr>
<td>O'Loughlin et al. (2004)</td>
<td>Centrifuge</td>
<td>L = 15 m D = 1.2 m 0 flukes</td>
<td>1315</td>
<td>1.7 - 3.6</td>
<td>2.0 - 2.9</td>
<td>1.2 - 1.5z</td>
</tr>
<tr>
<td>Richardson et al. (2009)</td>
<td>Centrifuge</td>
<td>L = 15 m D = 1.2 m 4 wide flukes</td>
<td>753</td>
<td>1.6 – 3.9</td>
<td>1.4 – 1.5</td>
<td>1.1z</td>
</tr>
<tr>
<td>de Sousa et al. (2011)</td>
<td>Numerical</td>
<td>L = 17.2 m D = 1.1 m 4 narrow flukes L = 0.75 m</td>
<td>1115</td>
<td>4.6</td>
<td>1.93</td>
<td>1.5z</td>
</tr>
<tr>
<td>This study</td>
<td>Field</td>
<td>D = 0.06 m 4 wide flukes</td>
<td>0.203</td>
<td>2.4 - 4</td>
<td>1.5 - 2.6</td>
<td>1.5z (z ≤ 1.5 m) 0.8z (z &gt; 1.5 m)</td>
</tr>
</tbody>
</table>
### Table 7.3 Input data for example application

<table>
<thead>
<tr>
<th>Soil</th>
<th>Undrained shear strength at mudline, $s_u$ (kPa)</th>
<th>0</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Shear strength gradient, $k$ (kPa/m)</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>Submerged unit weight, $\gamma'$ (kN/m$^3$)</td>
<td>6</td>
</tr>
<tr>
<td>Chain</td>
<td>Chain angle at mudline, $\theta_m$ (°)</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td>Nominal chain diameter, $d$ (m)</td>
<td>0.076</td>
</tr>
<tr>
<td></td>
<td>Effective width multiplier, $E_s$ (-)</td>
<td>2.5</td>
</tr>
<tr>
<td></td>
<td>Bearing capacity factor for the chain, $N_{c,\text{chain}}$ (-)</td>
<td>7.5</td>
</tr>
<tr>
<td></td>
<td>Chain-soil friction coefficient, $\mu$ (-)</td>
<td>0.1</td>
</tr>
<tr>
<td>Anchor</td>
<td>Bearing capacity factor, $N_i$ (-)</td>
<td>Tip</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Padeye</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Flukes</td>
</tr>
<tr>
<td></td>
<td>Interface friction ratio, $\alpha$ (-)</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Impact velocity, $v_i$ (m/s)</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>Factored design load at the anchor padeye, $F_i$ (MN)</td>
<td>5</td>
</tr>
</tbody>
</table>
### Chapter 7

**The University of Western Australia**

**Table 7.4 Test data for vertical loading**

<table>
<thead>
<tr>
<th>Drop reference</th>
<th>Drop height (m)</th>
<th>Impact velocity (m/s)</th>
<th>Anchor tip embedment, (z_e) (m)</th>
<th>Ultimate vertical capacity, (V_{ult}) (N)</th>
<th>Mooring line displacement required to mobilise maximum capacity, (\Delta) (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V1D1</td>
<td>0.000</td>
<td>0.000</td>
<td>1.120</td>
<td>518.51</td>
<td>-</td>
</tr>
<tr>
<td>V1D2</td>
<td>0.444</td>
<td>2.584</td>
<td>1.230</td>
<td>560.30</td>
<td>-</td>
</tr>
<tr>
<td>V1D3</td>
<td>1.016</td>
<td>3.758</td>
<td>1.473</td>
<td>600.90</td>
<td>-</td>
</tr>
<tr>
<td>V1D4</td>
<td>1.465</td>
<td>4.436</td>
<td>1.614</td>
<td>629.57</td>
<td>-</td>
</tr>
<tr>
<td>V1D5</td>
<td>2.755</td>
<td>5.562</td>
<td>1.832</td>
<td>691.75</td>
<td>-</td>
</tr>
<tr>
<td>V1D6</td>
<td>3.110</td>
<td>5.800</td>
<td>1.755</td>
<td>708.57</td>
<td>-</td>
</tr>
<tr>
<td>V2D1</td>
<td>3.513</td>
<td>5.973</td>
<td>1.785</td>
<td>707.37</td>
<td>-</td>
</tr>
<tr>
<td>V2D2</td>
<td>3.990</td>
<td>6.301</td>
<td>1.907</td>
<td>714.97</td>
<td>-</td>
</tr>
<tr>
<td>V2D3</td>
<td>4.528</td>
<td>6.299</td>
<td>1.823</td>
<td>702.19</td>
<td>-</td>
</tr>
<tr>
<td>V2D4</td>
<td>5.065</td>
<td>6.501</td>
<td>1.729</td>
<td>680.21</td>
<td>-</td>
</tr>
<tr>
<td>V2D5</td>
<td>1.581</td>
<td>4.627</td>
<td>1.634</td>
<td>627.92</td>
<td>-</td>
</tr>
<tr>
<td>V2D6</td>
<td>1.053</td>
<td>3.853</td>
<td>1.484</td>
<td>619.02</td>
<td>-</td>
</tr>
<tr>
<td>V2D7</td>
<td>0.696</td>
<td>3.203</td>
<td>1.388</td>
<td>581.41</td>
<td>-</td>
</tr>
<tr>
<td>V2D8</td>
<td>0.000</td>
<td>0.000</td>
<td>1.267</td>
<td>511.63</td>
<td>-</td>
</tr>
<tr>
<td>V3D1</td>
<td>0.000</td>
<td>0.000</td>
<td>1.231</td>
<td>503.83</td>
<td>-</td>
</tr>
<tr>
<td>V4D1</td>
<td>2.500</td>
<td>-</td>
<td>1.810</td>
<td>805.45</td>
<td>-</td>
</tr>
<tr>
<td>V4D2</td>
<td>2.500</td>
<td>-</td>
<td>1.860</td>
<td>786.11</td>
<td>0.276</td>
</tr>
<tr>
<td>V4D3</td>
<td>0.000</td>
<td>0.000</td>
<td>1.295</td>
<td>535.57</td>
<td>0.180</td>
</tr>
<tr>
<td>V4D4</td>
<td>0.000</td>
<td>0.000</td>
<td>1.390</td>
<td>529.43</td>
<td>0.193</td>
</tr>
<tr>
<td>V5D1</td>
<td>4.500</td>
<td>-</td>
<td>1.955</td>
<td>822.86</td>
<td>0.401</td>
</tr>
<tr>
<td>V5D2</td>
<td>4.500</td>
<td>-</td>
<td>1.915</td>
<td>823.09</td>
<td>0.308</td>
</tr>
<tr>
<td>V5D3</td>
<td>4.000</td>
<td>-</td>
<td>1.880</td>
<td>783.37</td>
<td>0.260</td>
</tr>
<tr>
<td>V5D4</td>
<td>4.000</td>
<td>-</td>
<td>1.885</td>
<td>769.37</td>
<td>0.296</td>
</tr>
<tr>
<td>V5D5</td>
<td>0.000</td>
<td>0.000</td>
<td>1.160</td>
<td>503.05</td>
<td>0.222</td>
</tr>
<tr>
<td>V6D1</td>
<td>0.000</td>
<td>0.000</td>
<td>1.130</td>
<td>515.71</td>
<td>0.270</td>
</tr>
<tr>
<td>V6D2</td>
<td>0.000</td>
<td>0.000</td>
<td>1.285</td>
<td>506.40</td>
<td>0.251</td>
</tr>
<tr>
<td>V6D3</td>
<td>1.000</td>
<td>-</td>
<td>1.550</td>
<td>598.38</td>
<td>0.239</td>
</tr>
<tr>
<td>V6D4</td>
<td>1.000</td>
<td>-</td>
<td>1.470</td>
<td>609.89</td>
<td>0.248</td>
</tr>
<tr>
<td>V6D5</td>
<td>2.000</td>
<td>-</td>
<td>1.695</td>
<td>682.89</td>
<td>0.289</td>
</tr>
<tr>
<td>V6D6</td>
<td>2.000</td>
<td>-</td>
<td>1.640</td>
<td>694.11</td>
<td>0.271</td>
</tr>
<tr>
<td>V6D7</td>
<td>2.860</td>
<td>-</td>
<td>1.830</td>
<td>735.05</td>
<td>0.289</td>
</tr>
</tbody>
</table>
### Table 7.4 Test data for vertical loading (continued)

<table>
<thead>
<tr>
<th>Drop reference</th>
<th>Drop height (m)</th>
<th>Impact velocity (m/s)</th>
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Table 7.5 Test data for inclined loading

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7.8 Figures

Figure 7.1 Dynamically installed anchors: (a) Deep Penetrating Anchor (Deep Sea Anchors 2009), (b) installation procedure

Figure 7.2 Undrained shear strength profiles
Figure 7.3 Model anchor
Figure 7.4 Field testing arrangement for: (a) vertical loading and (b) inclined loading
Figure 7.5 Load displacement response during vertical loading
Figure 7.6 Peak anchor vertical capacity as a function of initial embedment depth
Figure 7.7 Load displacement response during inclined loading
Figure 7.8 Load-displacement response from finite element analyses

Figure 7.9 Anchor capacity as a function of load inclination
Figure 7.10 Incremental soil displacements for: (a) $\theta = 90^\circ$, (b) $\theta = 60^\circ$, (c) $\theta = 45^\circ$, (d) $\theta = 0^\circ$, (e) pure rotation about the centroid

Figure 7.11 Applied load at the anchor padeye and the resulting loads at the anchor load reference point, located at the anchor centroid
Figure 7.12 V-M load interaction
Chapter 8  Conclusions

This chapter briefly concludes the full thesis by:

1. Recapping some of the main shortcomings concerning dynamically installed anchors which existed prior to undertaking this research. These shortcomings, discussed in Chapter 2, were important in that they led to the formulation of the main objective and corresponding sub-objectives of the research.

2. Explicitly discussing the achievement of each of the four sub-objectives as set out in the introduction (and thereby the achievement of the main objective of the research).

8.1 Main shortcomings existing prior to the research

Dynamically installed anchors seem to offer a cost effective and uncomplicated mooring solution to the offshore energy industry. Prior to this research however, significant uncertainty regarding their performance existed. Anchor capacity in deep-sea soils (where shear strengths typically increase with depth) depends largely on the final embedment depths achieved through the free-fall installation process. This depth can be difficult to predict due to the very high penetration velocities (up to ~30 m/s) that yield complex resistance forces. Chapter 2 highlighted a lack of comprehensive experimental data and field experience in the literature that could be used to verify the merit of existing predictive tools and to develop more comprehensive design procedures.

An analytical theoretical embedment model (refer to section 2.4) was a popular means of predicting the final embedment depth of high speed projectiles in clay. The forces that the model must compute iteratively, particularly strain-rate-dependent shearing resistance and fluid mechanics drag resistance, are complex and variations on the inclusion and formulation of individual resistance forces in the model existed across the literature.
Experimental studies on anchor installation which could be used to calibrate and validate the model had typically been at laboratory scale, considering only the response of the anchor during dynamic embedment in soil and ignoring the response of the anchor as it descends through the water column. Furthermore, the soil embedment data were generally limited to known starting and end conditions rather than continuous motion data. Field studies were rare and often did not include the level of seabed characterisation or anchor instrumentation necessary to validate prediction methods.

The soft soil response during high speed installations, specifically the hole created in the wake of an advancing projectile was poorly understood (refer to section 2.5); with conflicting hypotheses made across various studies. Whether the hole remains open or not will have an effect on the resistance forces acting during dynamic penetration (thereby affecting final embedment depths) and subsequent capacity calculations.

A simple technique for approximating the ultimate vertical tensile capacity of offshore driven piles in clay (API 2002, refer to section 2.6) is often used to calculate the capacity of dynamically installed anchors. Some disparity existed in the technique’s application with studies employing different interface friction ratios in the calculations (without adequate explanation) to ensure congruency with experimental results. More importantly, literature regarding the anchor’s response to inclined loading was extremely scarce, representing a major shortcoming considering that mooring lines typically arrive at an angle to the anchor padeye.

8.2 Achieving the sub-objectives as set out in Chapter 1

1. Develop an experimental database, encompassing both centrifuge and field tests, for dynamically installed anchors as a means of establishing anchor performance in soft soil and for calibrating design tools.

Experimental studies involving reduced scale field tests and centrifuge tests on a typical dynamically installed anchor geometry were successfully completed. The testing programs (together comprising over 200 tests) have led to a broad accessible database, representing a significant contribution to the offshore engineering community. Designers and workers now have a greatly improved means of calibrating numerical models and verifying calculations.

The anchor geometry tested was based on an idealised design proposed by Lieng et al. (1999), featuring four clipped delta type flukes (separated by 90° in plan) with a forward
swept trailing edge and an ellipsoidal tip. The field tests were carried out in a soft soil lake environment in Northern Ireland using a 1:20 scale (750 mm long) anchor in water depths of 3 to 18 m over a two year period from 2010 to 2012. These field tests represent a valuable contribution to the literature on dynamically installed anchors. The centrifuge tests were carried out using a 75 mm long model anchor, along with flukeless projectiles of varying aspect ratio, in the beam centrifuge facilities at the Centre for Offshore Foundation Systems, UWA over a one and a half year period from 2013 to 2014. These tests captured full motion history data and high speed video imagery of the soft soil response during installations in normally and overconsolidated clay. This denotes a significant advancement on preceding centrifuge test data for dynamically installed anchors that were primarily restricted to known starting and end conditions (the measured anchor impact velocity and measured anchor final embedment depth) in normally consolidated clay.

2. Answer the question as to how the soil responds in the wake of a dynamically installed anchor during dynamic embedment, establishing the conditions under which the hole formed by the passage of the anchor may close.

Significant understanding of the soil response in the wake of dynamically installed anchors, and indeed dynamic projectiles at large, during high-speed penetration was made through analyses of the centrifuge test data. This was the focus of Chapter 3 in the thesis. An video camera arrangement set to record at a frame rate of 2.27 kHz in-flight captured the impact and early penetration event of numerous dynamic projectiles in soil samples with varying mudline strengths and overconsolidation ratios.

Whether the hole closed or remained open, either fully or partially was found to be dependent on the dimensionless strength ratio, $s_u/\gamma'D$ at the rear of the at-rest projectile. It was experimentally shown for the first time that when any hole closure does occur, it may do so at the same rate as the high speed projectile penetrates, i.e. whilst the projectile is penetrating and not after it comes to rest. This closure mechanism was found to be controlled by soil backflow at the rear of the projectile, indicating shallower cavity depths than would be predicted using common wall failure criterion. By virtue of testing for a range of dynamic projectiles and not only the model anchor, the findings were shown to be consistent for projectiles of varying aspect ratio.
A framework for predicting whether the hole closes or remains open by calculating the transitional depth for backflow failure was given. In Chapter 4, using the framework to make the correct interpretation in an analytical embedment model was shown to provide better agreement with experimental motion profiles than for incorrect interpretation being made. The framework was again used in Chapter 6 for correct interpretation of field installation data. This framework will aid in the design and modelling of dynamically installed anchors through more methodical penetration and capacity analyses.

3. **Refine and calibrate analytical embedment models for dynamically installed anchors using motion data collected in the centrifuge and field tests from sub-objective one.**

For both the field and centrifuge installation tests, the model anchors were instrumented with accelerometers and (for the field tests) rate gyroscopes that allowed a continuous time history of the anchor’s motion to be established over the full dynamic event (free-fall through water or air and soil penetration). Considering the novelty of the inertial measurement unit (IMU) used in the field, its technical development was described in detail in Chapter 5 together with the merit of a comprehensive framework for interpreting the recorded motion data. The micro-electro mechanical system (MEMS) accelerometer used in the centrifuge installations was discussed in Chapter 4 with a more detailed review of using the device in dynamic centrifuge tests available in O’Loughlin et al. (2014b).

The motion data captured the acceleration of the anchors relative to the mudline, thus allowing for rigorous assessment of resistance forces acting on the anchor not only during soil penetration but also prior to soil impact. This is an important consideration for correct calibration of embedment models that are based on the interpretation of projectile net force. Previous experimental studies have primarily been restricted to impact velocity measurements. This may not give an accurate depiction of net force as the resistance forces acting during free-fall which continue to act during initial soil penetration (i.e. water drag resistance of the anchor and its trailing line in the field, and guide friction in the centrifuge) cannot be accurately quantified. The research presented in the thesis has overcome such limitations.
Adhering to the soil response findings of Chapter 3 (and as a direct result of the MEMS motion data) Chapter 4 indicated that current strain rate relationships used in an analytical embedment model become inaccurate over several orders of magnitude increase from the reference strain rate; with mobilised soil shear strengths increasing more rapidly than can be fitted using a common power law. This led to the formulation of a new equation for varying the strain rate factor so that the power function may be applied to the very high strain rates observed in dynamic centrifuge tests. The aptness of including drag resistance during soil penetration in the embedment model was verified.

Chapter 6 presented field installation data gathered through sub-objective 1 along with analyses of an additional smaller suite of tests reported by industry using a 1:3 scale dynamically installed anchor in the North Sea. The combined database of 114 anchor installations formed the basis for a new Release-to-Rest embedment model which was validated against the motion history data recorded at both sites. This model extends on existing embedment prediction tools by considering the motion of the anchor from the point of release in the water column, modelling the drag resistance that acts on the anchor and its mooring line; the latter shown to be significant for larger release heights, reducing the anchor velocity as it arrives at the seabed by up to 44%. The R2R model can be readily implemented – either on a spreadsheet or using a programming language – making it a useful tool for practitioners.

Chapter 4 and 6 also considered the potential for higher strain rate dependence on frictional resistance than on bearing resistance in the embedment model which has been suggested by a number of other dynamic studies. This approach was shown to give exactly equivalent predictions (in both the centrifuge and field datasets) as when the same strain rate dependence was modelled for bearing and friction, but required an interface friction ratio that is lower than the inverse of the soil sensitivity. This reduced interface friction ratio is consistent with water entrainment along the anchor-soil interface that would mobilise a lower (reference) soil strength than the remoulded soil strength, leading to lower frictional resistance.
4. Establish design tools for predicting the monotonic capacity of dynamically installed anchors for a range of load inclinations, calibrated using field data from sub-objective one.

Chapter 7 described findings made through analyses of data recovered from 89 monotonic loading field tests (sub-objective 1) and additional corresponding large deformation finite element (LDFE) analyses. Ultimate vertical capacities in the field were between 2.4 and 4.1 times the anchor dry weight reflecting tip embedment depths, achieved through dynamic installation, of 1.5 to 2.6 times the anchor length. The merit of the American Petroleum Institute (API) approach for estimating the vertical capacity of driven piles was systematically verified against this field data using an interface friction ratio of 0.5, which is consistent with mobilisation of the remoulded soil strength, and reflects the negligible reconsolidation time allowed for in the tests. This verification was a key prerequisite for developing the design tool encompassing inclined loading given towards the end of the chapter.

The anchor capacity and corresponding padeye displacement required to mobilise it were found to increase under inclined loading. The field data indicated a capacity increase as high as 30% for the minimum achievable inclination of about 20 degrees to the horizontal and a corresponding mobilisation displacement of almost 3 times that required for vertical loading. Results from the numerical analyses were consistent with the experimental data, although actual capacities were overestimated due to a higher interface friction ratio of 1 being adopted; a reduced value leading to non-convergence or instability in the Lagrangian calculations. Maximum anchor capacities resolved through the numerical analyses were at load inclinations of between 30° and 45° to the horizontal, and were approximately 20% higher than for pure vertical loading.

The interaction between vertical and horizontal (and hence moment) capacities was explored using the LDFE analyses and a resulting yield envelope was found to be in good agreement with the experimental data. This was used as the basis for establishing a simple design procedure for scaling a dynamically installed anchor for a given design load over load inclinations ranging from purely vertical to purely horizontal loading. The procedure is applicable for the specific anchor geometry considered in this thesis and similar approaches may be developed for other geometries by following the analyses described within Chapter 7.


Acteon. 2009. Petrobras awards exclusive torpedo pile technology rights to InterMoor. The acteon customer magazine, 6(9), p. 5.


References


References

Modelling. Civil Engineering. Campos dos Goytacazes, Brazil: State University of Norte Fluminense Darcy Ribeiro - UENF.


References


245


References


References


Schmid, W.E. 1969. Penetration of objects into the ocean bottom, Report No. AD 695434, Naval Civil Engineering Laboratory, Port Hueneme, California.

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References


