Centrifuge Modelling and Numerical Analysis of Penetrometers in Uniform and Layered Clays

By

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B.Eng.

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Doctor of Philosophy

of The University of Western Australia

School of Civil, Environmental and Mining Engineering
Centre for Offshore Foundation Systems

April 2019
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ABSTRACT

Increasing application of penetrometer tests, including cone and full-flow penetrometers i.e. T-bar and ball, in offshore site investigations has been muddied by the lack of accurate interpretation framework for their resistance profiles, especially in layered seabed sediments. This research has focused on (i) revealing the evolving soil failure mechanisms around the T-bar, ball and cone penetrometers during their continuous penetration into layered fine-grained sediments from the surface/mudline; and (ii) quantifying their effects on the resistance profiles. The overarching aim was to establish a design framework for interpreting undrained shear strength from the measured penetration resistance profiles through offshore in-situ site investigations.

To reveal the soil failure mechanism, a series of centrifuge tests was conducted with the penetrometers penetrating through layered clays, such as soft-stiff, stiff-soft, soft-stiff-soft, and stiff-soft-stiff clay deposits. Particle image velocimetry (PIV) allowed accurate resolution of the flow mechanism around a face of the T-bar, half-ball and half-cone penetrating adjacent to a transparent window. To quantify the layering effect, an extensive parametric study by large deformation finite element (LDFE) analysis was conducted for the T-bar, since cone and ball have been analysed numerically previously.

In centrifuge tests, for T-bar, overall a symmetric rotational flow around the T-bar dominated the behaviour. A novel ‘trapped cavity mechanism’ was revealed in stiff clays, with its formation and closure tracked. For the ball, the key features of soil flow include vertical flow, cavity expansion and rotational flow. For both full-flow penetrometers, a squeezing mechanism dominated the behaviour when approaching the soft-stiff interface, with soft soil squeezed sideways before involving the bottom
stiff soil, leading to minimal deformation of the layer interface. In a stiff-soft clay deposit, a ‘punch-through’ type of failure occurred near the layer interface with large downward deformation of the layer interface. The effect of layering was also quantified through comparing trajectories of soil element at different distances from the interface and penetrometers. A stiff clay plug was trapped at the base of the ball, which was forced into the softer underlying layer with the ball. However, there was no soil plug observed from the previous layer trapped at the base of the advancing T-bar regardless of penetration from stiff to soft clay or the reverse. For the cone in a single uniform clay layer, based on the centrifuge test results, the existing strain path method showed reasonable predictions on maximum lateral and vertical displacements in the soil around the cone shoulder. However, the upheave movement in soil was overestimated. For the cone in layered soils, the squeezing mechanism at soft-stiff layer interface and ‘punch-through’ failure mechanism at stiff-soft layer interface was also observed. Due to the shape of the cone tip, there was no trapped soil from the previous layer underneath the cone tip when entering the underlying layer in stiff-soft clays or vice versa.

The penetration resistance profiles in the layered sediments were obtained by centrifuge tests using separate miniature T-bar, full-ball and full-cone. The resistance profiles were linked to the mobilised soil failure mechanisms at various penetration stages. A novel calibration method on cone tip load cell under ambient pressure was proposed for proper conversion of the cone resistance measured in soils.

In LDFE analysis, the behaviour and T-bar resistance factor profile in both single- and two-layer clay have been studied, with a T-bar penetrating continuously from the mudline into both single and layered profiles. For single layer of clays, the formation of the trapped cavity above the advancing T-bar and its evolution to full flow
mechanism was studied extensively, covering a large range of soil normalised undrained shear strength and the T-bar surface roughness. It was found that the depths of trapped cavity formation and closure increased with increasing normalised undrained shear strength and T-bar roughness. The presence of the trapped cavity could reduce the T-bar bearing capacity factor that was used in current practice, where a full flow mechanism was assumed. For two-layer clays, the effect of soil layering on the behaviour of the T-bar penetrometer was explored in both soft-stiff and stiff-soft clay deposits. The resistance profiles were discussed linking with the mobilised soil failure mechanisms. Finally, based on the systematic parametric analyses performed, the interpretation framework accounting for the effect of the trapped cavity and layering was proposed for more accurate interpretation of soil undrained shear strength and soil interface elevations.

The soil flow mechanism and shearing around the ends of the T-bar cylinder, termed as ‘end effect’, was investigated with a three-dimensional finite element model (3D-FE), with varying T-bar aspect ratio (length/diameter) and surface roughness. A practical correction method accounting for the end effect was proposed to adjust the commonly used analytical plane strain solutions.
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NOTATIONS

\( a \)  
shaft-ball area ratio

\( A \)  
projected area of T-bar

\( A_s \)  
submerged cross-sectional area of T-bar

\( c_v \)  
coefficient of consolidation

\( C \)  
calibration factor

\( C_{ax} \)  
calibration factor from axial load test

\( C_{am} \)  
calibration factor from hydraulic test

\( d \)  
penetration depth from soil surface (measured from penetrometer tip/invert)

\( D \)  
diameter of T-bar in Chapter 5-8

\( D_b \)  
diameter of ball penetrometer

\( D_c \)  
diameter of cone penetrometer

\( D_t \)  
diameter of T-bar in Chapter 1-3

\( D_s \)  
diameter of shaft of ball penetrometer

\( E \)  
Young’s modules

\( f_b \)  
buoyancy factor for local heave

\( F \)  
measure tip penetration force of penetrometers

\( F_b \)  
buoyancy force of T-bar

\( F_{in} \)  
the increase of resistance factor in bottom layer of stiff-soft clay

\( F_{net} \)  
et resistance force of unit length T-bar
\( h \)  layer thickness \((h_1; h_2; h_3 \text{ for corresponding layer})\)

\( h_{\text{min}} \) minimum mesh size

\( h_t \) normalised penetration depth of forming trapped cavity

\( h_{tf} \) normalised penetration depth of fully backfilling trapped cavity or appearing fully localised flow-round mechanism

\( k \) strength gradient

\( L \) length of T-bar cylinder

\( N_b \) resistance factor of ball penetrometer

\( N_c \) resistance factor of cone penetrometer

\( N_{cf} \) resistance factor of T-bar at the start of trapped cavity mechanism

\( N_{ff} \) resistance factor of T-bar with fully localised flow-round mechanism (stage III; with trapped cavity)

\( N_p \) resistance factor of the penetrometer

\( N_{sf} \) resistance factor of T-bar during shallow failure mechanism (stage I; with open cavity)

\( N_t \) resistance factor of T-bar penetrometer

\( N_{te} \) resistance factor of T-bar during trapped cavity mechanism (stage II; with trapped cavity)

\( N_{t,b} \) resistance factor of T-bar in the bottom layer of stiff-soft clay

\( N_{t,u} \) resistance factor of T-bar using the strength of soft bottom layer of stiff-soft clay

\( N_{t,3D} \) resistance factor of T-bar penetrometer considering end effect

xxx
$N_{\text{ideal}}$ resistance factor of T-bar penetrometer in plane strain condition

$q$ measured tip resistance of penetrometer

$q_{\text{net}}$ net resistance of penetrometer

$r$ radius of T-bar

$S_t$ sensitivity

$s_u$ undrained shear strength of soil ($s_{u1}$; $s_{u2}$; $s_{u3}$ for corresponding layer)

$s_u(\text{CPT})$ undrained shear strength of soil interpreted from cone penetrometer

$s_u(\text{ball})$ undrained shear strength of soil interpreted from cone penetrometer

$t$ vertical distance from penetrometer tip to layer interface

$v$ penetration velocity of penetrometers (model scale)

$V$ dimensionless velocity

$v_t$ voltage response of penetrometer

$v_{\text{soil particle}}/v_{T-bar}$ penetration rate of traced soil element relative to T-bar penetration

$x$ horizontal distance from centreline of penetrometer in Chapter 2-3

$x$ horizontal distance from T-bar end in Chapter 7

$y$ vertical distance from the tracked element (downward direction positive)

$y_1$ depth of cone at start of soil trajectory tracking (in corresponding to tracked element)

$y_2$ depth of cone at the end of soil trajectory tracking (in corresponding to tracked element)

$\alpha$ roughness factor
\( \beta \)  contribution of the end effect

\( \gamma \)  total unit weight of soil

\( \gamma' \)  submerged unit weight of soil (\( \gamma'_1; \gamma'_2; \gamma'_3 \) for corresponding layer)

\( \Delta d \)  penetration depth interval

\( \Delta x \)  lateral displacement of soil element

\( \Delta y \)  vertical displacement of soil element

\( \sigma_{vo} \)  overburden pressure of cone penetrometer

\( \mu_C \)  Coulomb friction coefficient

\( \tau_{\text{max}} \)  the limiting shear stress of soil
<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
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<tr>
<td>2D</td>
<td>Two-dimensional</td>
</tr>
<tr>
<td>3D</td>
<td>Three-dimensional</td>
</tr>
<tr>
<td>ALE</td>
<td>Arbitrary Lagrangian-Eulerian</td>
</tr>
<tr>
<td>CPT</td>
<td>Cone penetration test</td>
</tr>
<tr>
<td>FE</td>
<td>Finite element</td>
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<tr>
<td>LDFE</td>
<td>Large deformation finite element</td>
</tr>
<tr>
<td>PIV</td>
<td>Particle image velocimetry</td>
</tr>
<tr>
<td>RITSS</td>
<td>Remeshing and interpolation technique with small strain</td>
</tr>
<tr>
<td>SPM</td>
<td>Strain path method</td>
</tr>
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<td>Shallow strain path method</td>
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CHAPTER 1  INTRODUCTION

1.1 In-situ investigations

Appropriate design approaches or prediction methods are the cornerstones for safe and economic designs of offshore pipelines, foundations and anchoring systems. Under various design codes (e.g. ISO, API, DNV) and joint industry projects (e.g. InSafeJIP), existing design approaches are being improved continuously based on the large database of case histories in publications. However, soil site investigations provide essential input parameters for foundation designs. Hence, it is vitally important to improve site investigation standards, identifying the most appropriate forms of in-situ and laboratory testing and interpretation method to select suitable soil strength parameters for designs.

Recent exploration of hydrocarbon products in deep and ultra deep (in excess of 3000 m) water depths and remote locations has placed more reliance on in-situ testing. Figure 1-1 shows some examples of single and layered seabed soil profiles from the Gulf of Mexico, Sunda Shelf, offshore India and Australia (Handidjaja et al., 2004; Erbrich, 2005; Watson & Humpheson, 2005; Kostelnik et al., 2007; Menzies & Roper, 2008; Purwana et al., 2009; Teh et al., 2009; Menzies & Lopez, 2011). The data from continuous penetrometer profiles are essential, both in terms of identifying any anomalous soil layers, and in providing initial guidance on the estimation of foundation/anchor penetration resistance. This is critical for offshore foundation/anchoring systems ranging from shallow depths’ pipeline, mudmat, skirted or bucket foundations to deeper depths’ piles, suction caissons to spudcan foundations,
dynamically installed anchors. For instance, suction caisson’s lid remains in the mudline but the skirt tip penetrates up to 30 to 40 m below the mudline, and spudcan penetrates continuously up to 30–50 m in clayey seabed sediments. Therefore, the need to include in-situ tests in the site investigation program for each offshore project has been repeatedly emphasised by the offshore industry (API, 2002; Paisley & Chan, 2006; Menzies & Lopez, 2011; Osborne et al., 2011).

In layered sediments, the identification of the layer boundaries and selection of soil strength parameters for each identified layer are the key procedures based on the in-situ site investigation data. This is because the adjacent layers can affect the resistance profile in the current layer in in-situ site investigation data. However, the majority of well-established interpretation methods are for a single soil layer. The rational identifications of the layer boundaries and soil strength parameters of each layer underpin the accuracy of the estimated foundation/anchor resistance profile during installation and capacity under operational loadings. Hence it is an integral part of improved designs to refine and improve the frameworks for interpreting layer boundaries and shear strength from in-situ site investigation data in layered deposits.

1.2 In-situ penetrometers

There is a wide range of in-situ site investigation tools (see Figure 1-2) to choose from depending on the aim of the site investigation. Vane shear and cone penetrometer are the most commonly used tools for measuring soil shear strength. The relatively recent inclusions are T-bar and ball penetrometers, which are mostly applicable for clayey sediments. Compared to vane shear test, push-in penetrometers (cone, T-bar, ball) generally provide continuous or near-continuous resistance profiles and hence higher
resolution of soil strength profiles and layer interfaces. Figure 1-3 shows the platforms for carrying out onshore and offshore push-in penetrometer tests.

The dimensions of three push-in penetrometers have been illustrated in Figure 1-4. Cone penetrometer test (CPT) has gained wide acceptance for characterisation of soils due to the reliability and repeatability of the measurements (Lunne et al., 1997). Cone penetrometers are cylindrical in shape with a conical tip. Although the size varies in the field, the standard cone penetrometer has a base area of 10 cm$^2$ cone (diameter = 35.7 mm) and a 60° tip-apex angle (Figure 1-4). Along with the tip load cell, friction sleeves, face and shoulder pore pressure transducers can be embedded to the cone to determine soil friction, classification, permeability and other properties. Cone penetrometers are also used for soil characterisation in physical model testing in laboratory floor (i.e. 1g) tests or in centrifuge (enhanced gravity) test, where the diameter of the miniature model cone penetrometers vary from 7 mm to 10 mm (Lee, 2009). The standard procedure of cone penetrometer test can be found in Robertson & Cabal (2015).

The T-bar and ball penetrometers (Figure 1-4) are referred as ‘full-flow’ penetrometer as they allow soil to flow around the probe as it penetrates into soils, in contrast to the cone where the adjacent soil is displaced outward to accommodate the cylindrical shaft. The commonly used T-bar penetrometer is 40 mm in diameter and 250 mm in length (projection area = 100 cm$^2$), with a shaft connected to the centre of the bar. The ball penetrometer is 113 mm in diameter (projection area = 100 cm$^2$). A pore pressure transducer is sometimes installed either at the base face or on the equator of the ball sphere to evaluate the permeability of soils. The T-bar and ball penetrometers have been proven to be superior to the cone penetrometer for offshore site investigations (at least for clayey sediments) in the following aspects (Low, 2009):
(1) relatively large projected area (5~10 times larger than cone), providing high resolution of penetration resistance profiles in soft sediments (~1kPa), which are frequently encountered in deep water sites (Randolph et al., 1998; Hefer & Neubecker, 1999);

(2) relatively small shaft-penetrometer area ratio, producing negligible influence of overburden pressure to the total resistance i.e. measured resistance \( q = \text{total resistance} = \text{net resistance} \ q_{\text{net}} \); and

(3) negligible influence of soil rigidity index on the measured resistance or resistance factor (Lu et al., 2000).

It should be also noted that the large projection area of T-bar and ball penetrometer would lead to large penetration force therefore limit the investigation depth. Similarly, the sand layers in sediments might lead to T-bar refusal or bending issue. Yet, full-flow penetrometer is still an advantageous member to the penetrometer family.

1.3 Background

This study focuses on strength profiles of clayey or fine-grained seabed sediments. The undrained shear strength profile of fine-grained seabed sediments is interpreted from penetration resistance profiles of penetrometers according to

\[
 s_u = \frac{q_{\text{net}}}{N_p} \tag{1-1}
\]

where \( s_u \) is the undrained shear strength of soil, \( q_{\text{net}} \) is the tip net resistance and \( N_p \) is the resistance factor of the penetrometer (\( N_t \) for T-bar, \( N_b \) for ball and \( N_c \) for cone).

For the T-bar and ball penetrometers, as noted previously, the measured resistance \( q = \text{total resistance} = \text{net resistance} \ q_{\text{net}} \), and for the cone penetrometer, the measured tip
resistance is firstly corrected to total cone resistance (for the piezocone only) and then the net resistance $q_{\text{net}}$ is calculated through deducting in-situ total overburden stress from the total resistance. Essentially, in Equation 1-1, only input is the value of resistance factor, $N_p$. The resistance factors are back calculated calibrating penetrometer’s net resistance against the measured intermittent shear strength profile from laboratory tests on cored samples. However, ideally, in-situ testing methods should be developed to a stage where design parameters may be obtained with minimal (or ultimately no) requirement for calibration at each site by means of laboratory test data (Low et al., 2010). For fulfilling that aim, penetrometers’ resistance factors have been investigated extensively through both theoretical solutions and numerical analyses.

### 1.3.1 Theoretical solutions

The theoretical solutions of $N_p$ rely on assumed soil failure mechanisms. For the T-bar and ball penetrometers, a full flow-round mechanism for a plain strain cylindrical bar and axisymmetric ball respectively was assumed. The plasticity solutions resulted in resistance factors of $N_t = 9.1$–$11.9$ (Randolph & Houlsby, 1984; Martin & Randolph, 2006) for the cylinder bar and $N_b = 11.0$–$15.3$ for the spherical ball (Randolph et al., 2000) depending on the penetrometer’s surface roughness. For the cone penetrometer, the use of limit analysis is restricted because new intrusive volume is constantly introduced into the soil during the penetration process (Einav & Randolph, 2005). The theoretical estimation of resistance factor for the cone is mainly built on spherical cavity expansion theory (Teh & Houlsby, 1991) or strain path method (Baligh, 1985), which falls in the range of $10.0$–$17.5$ (Yu et al., 1998). However, the soil failure mechanisms evolve with the continuous penetration of these penetrometers, and the
existing analytical solutions based on a particular mechanism cannot represent the evolution.

### 1.3.2 Numerical analyses: single layer fine grained sediments

Large deformation finite element (LDFE) analysis has been advantageous in respect of removing the necessity of priori assumption of a soil failure mechanism; exposing the evolution of soil failure mechanisms and providing corresponding continuous penetration resistance and hence resistance factor (Wang et al., 2015). This section will be focusing on previous numerical studies of all three penetrometers in single layer fine grained sediments (which is prevalent in the Gulf of Mexico; Menzies & Roper, 2008; Hossain et al., 2015) before moving to stratified sediments.

**T-bar Penetrometer**

Pushing a plane strain cylindrical bar continuously from the soil surface, there are two mechanisms found by White et al. (2010): shallow mechanism with an open cavity above the bar and deep flow-round mechanism around the fully embedded bar. The depth of attaining the latter mechanism was shown to increase with increasing normalised undrained shear strength, \( s_u/\gamma'D_t \), where \( \gamma' \) is the submerged unit weight of the soil, and \( D_t \) is the diameter of the T-bar cylinder. The T-bar resistance profile was therefore divided into shallow (before mobilising the flow-round failure) and deep (after mobilising the flow-round failure) penetration stages. The soil strength interpretation framework based on these two penetration stages was proposed accordingly. However, Tho et al. (2011) identified a trapped cavity mechanism, where an enclosed cavity was found at a distance above the T-bar rather than just above the T-bar. In soils with \( s_u/\gamma'D_t > 3 \), the trapped cavity mechanism occurred following the shallow mechanisms for a smooth T-bar (i.e. roughness factor \( \alpha = 0 \)). Evidently, the
trapped cavity led a 12% lower T-bar resistance factor relative to the deep flow-round failure mechanism. For the standard T-bar penetrometer commonly used in offshore site investigations ($D_t = 0.04$ m) and the commonly encountered offshore clays with undrained shear strengths of $s_u = 5$~25 kPa, the normalised strength falls in the range of $s_u/γ'D_t = 20$~100, although shear strength of the seabed surficial sediment can be as soft as 0~5 kPa. The existing soil strength interpretation framework (White et al., 2010) was limited to $s_u/γ'D_t ≤ 10$ and did not take into account the trapped cavity mechanism. Tho et al. (2011) considered the trapped cavity, but was also limited to $s_u/γ'D_t ≤ 10$ and fully smooth T-bars (i.e. $α = 0$). Thus it is imperative to conduct a more systematic study to cover the practical range of $s_u/γ'D_t = 20$~100 for T-bar penetrometers with different roughness factors.

The T-bar penetrometers of various sizes are used in the field and particularly in centrifuge testing. The length to diameter ratio lies in the range of $L/D_t = 4$~6.25. The existing investigations mentioned above were either on pipelines or T-bar with an infinite length for plane strain conditions. The potential 3D effect of the T-bar end was not considered.

**Ball Penetrometer**

For the ball penetrometer (including the shaft) in a single layer clay, a shallow mechanism with an open cavity above the penetrating ball and a deep flow-round mechanism were reported by Zhou et al. (2013). LDFE analyses for a pre-embedded ball have been performed by Zhou & Randolph (2009, 2011). The effect of the shaft-ball area ratio ($a = D_s^2/D_b^2$; where $D_s$ is the shaft diameter and $D_b$ is the ball diameter) on the resistance factor $N_b$ was also explored by Zhou & Randolph (2011) and Zhou et al. (2013). It was found that the resistance factor $N_b$ for the ball with a shaft ($a > 0$)
was lower than that without a shaft \((a = 0)\). The combined mechanism of cavity
expansion and localised back flow around the embedded ball was observed for a ball
with a shaft, compared to the classic full-flow mechanism for a spherical ball. The
effect of ball surface roughness on the ball factor \(N_b\) was also investigated, leading to
no further study of the ball in single layer clay needed.

**Cone Penetrometer**

For the cone penetrometer penetration in single layer clay, LDFE analyses have been
carried out by notably Van den Berg *et al.* (1996), Liyanapathirana (2009), Lu (2004)
and Ma *et al.* (2014). The former three have focused on the deep resistance factor
 correponding to a mobilised cavity expansion type failure. Ma *et al.* (2014)
investigated the full penetration process of the cone penetrometer from the soil surface.
A new formula for calculating the stabilised cone bearing capacity factor was proposed,
accounting for the effect of soil rigidity index, soil strength non-homogeneity, in-situ
stress ratio and cone roughness. An interpretation framework was also proposed for
interpreting the cone penetrometer data at shallow penetration depths prior to
mobilising the stable deep resistance. Thus, no further study is needed.

1.3.3 Numerical analyses: stratified fine grained sediments

In order to comply with the global ever-increasing demands for energy, oil and gas
exploration is gradually shifting to deeper waters, and unexplored and undeveloped
provinces. A range of seabed stratifications are encountered in emerging fields such
as in the Sunda Shelf, offshore Malaysia, Australia’s Bass Strait and North-West Shelf,
Gulf of Thailand, South China Sea, Offshore India, and Arabian Gulf (Osborne *et al*.,
2011).

**T-bar Penetrometer**
Introduction

As of concern, no investigation was reported on the T-bar penetrometer penetration in layered soil deposits. It is therefore an urgent need to examine the influence of soil layering on the T-bar penetrometer behaviour, and to establish a design framework for interpreting undrained shear strength profile along with identifying layer boundaries.

**Ball Penetrometer**

The ball penetrometer (including a shaft) penetrating continuously from the surface through layered soft-stiff and stiff-soft clay deposits has been investigated by Zhou *et al.* (2013, 2016). The effect of soil layering on the mobilised failure mechanisms, and critically on trapping of a stiff soil plug beneath the advancing ball, was shown to be significant. A framework was proposed to account for this effect, along with the effects of the ball surface roughness and area ratio, for rational interpretation of layer boundary and undrained shear strength of each layer.

**Cone Penetrometer**

For the cone penetration in multilayer clays, Walker & Yu (2010) performed analyses in two layer stiff-soft and three-layer stiff-soft-stiff clay deposits. Ma *et al.* (2015, 2017a, 2017b) have explored a much wider range of soil layered profiles encompassing soft-stiff, stiff-soft, soft-stiff-soft and stiff-soft-stiff clay profiles. It was shown that (i) the undrained shear strength of a soft layer can be interpreted by using a single layer approach and the resistance profile without the influence of the adjacent stiff layer; (ii) the interpretation for the interbedded stiff layer necessitates implementing a correction factor as a function of the layer thickness, rigidity index of the stiff layer and the strength ratio between that layer and the bottom layer. Design frameworks have been proposed and illustrated through flowcharts to be used in practice.
1.3.4 Model testing: soil failure mechanisms

To validate the soil flow patterns exposed from continuous penetration LDFE analyses, it is of great interests to directly visualise the flow mechanisms during the penetration process of penetrometers. The soil flow mechanism around a penetrating model is mostly investigated with properly scaled model testing in the centrifuge as it reproduces the field stress condition, which is essential for mobilising true soil flow mechanisms such as soil back flow, deep flow-around mechanism.

Transparent soil substitutes have been developed (Gill & Lehane, 2001; Iskander et al., 2002) to assist the observation of flow mechanism of penetrometers, since the opacity of natural soils prevents observations of the soil motion during penetrometer penetration into the soil sample. Recently, half-model test against a window followed by particle image velocimetry (PIV) analyses has been a popular technique in centrifuge testing for revealing the true soil failure mechanisms (e.g. Hossain & Randolph, 2010; Hossain, 2014; Stanier et al., 2015). However, this technique has yet to be used for observing the soil failure mechanisms around penetrometers except the investigations on cone penetrometers in single layer and layered sands by Paniagua et al.’s (2013), Arshad et al.’s (2014) and Mo et al.’s (2015, 2017).

1.4 Aim of this research

The motivation and goals of this study are emanated directly from the ‘future needs’ and identified based on the background discussed above. Although all three penetrometers have been studied continuously, there are only limited studies reporting visualisations of soil flow mechanisms of penetrometers penetration into single and layered clays. Both cone and ball penetrometer have been studied extensively numerically in layered clays, but T-bar was only studied for low undrained shear
strength \( \frac{s_u}{\gamma'D_t} < 10 \) in single layer clay. Thus this thesis focuses on the revelation of the soil flow mechanism of all three penetrometers in layered sediments through centrifuge testing, with an extensive FE analysis on T-bar only. The selected key aims are:

(1) revealing the evolution of soil failure mechanisms associated with the continuous penetration of all three penetrometers (i.e. T-bar, ball and cone) in stratified clay deposits through model testing in a centrifuge;

(2) investigating the T-bar penetrometer penetration in single layer clay deposits with a practical range of normalised strength \( \frac{s_u}{\gamma'D_t} \) through LDFE analyses, and proposing an improved interpretation framework taking into account the effect of the trapped cavity mechanism;

(3) exploring the T-bar penetrometer penetration in two-layer stiff-soft and soft-stiff clay deposits through LDFE analyses, and proposing a new interpretation framework taking into account the effect of soil layering;

(4) examining the end effect of the T-bar penetrometer through three-dimensional (3D) small strain finite element analyses;

(5) practising a novel calibration method for the calibration of cone tip load cell to cope with the issue rising from the conversion of cone tip resistance to net resistance in centrifuge test.

1.5 Outline of this research

This thesis comprises eight chapters. Following the University of Western Australia’s regulations regarding to Research Higher Degree, this thesis is presented as a series of papers that has been published (Chapters 4, 7) or submitted (Chapters 2) or prepared
Chapter 1

for submission (Chapter 3, 5, and 6). A conclusive Chapter 8 summarises the major findings of the thesis and outlines potential future research directions. The methodology of this study includes centrifuge testing (Chapters 2~4), two-dimensional LDFE analysis (Chapters 5~6) and three-dimensional FE analysis (Chapter 7). The outline of this thesis is displayed below.

**Chapter 2** presents the results of PIV analysis of centrifuge tests on the T-bar and ball penetrometers in layered clay deposits. The soil failure mechanisms are revealed in soft-stiff, stiff-soft, soft-stiff-soft, and stiff-soft-stiff, and the effect of soil layering on the mobilised soil flow mechanisms is highlighted. The soil displacement paths induced by the penetration of the full-flow penetrometers are tracked, and the layered effect is discussed through the comparison of the displacement paths of soil elements at various locations, as functions of distances away from the layer interface and away from the centreline of the penetrometer penetration path.

**Chapter 3** presents the results of PIV analysis of centrifuge tests on the cone penetrometer in layered clays. The measurements quantified through PIV analyses are compared against those from the theoretical Shallow Strain Path Method (SSPM). The effect of soil layering on the failure mechanisms is revealed through the evolutions of both flow patterns and displacement paths at various distances from the layer interface.

**Chapter 4** presents a novel technique in calibrating ambient pressure on the cone tip load cell to ensure a correct conversion of overburden pressure. The load cell behaviours under axial load calibration test (1g test on the laboratory floor) and ambient pressure calibration test (hydraulic pressure tests in a centrifuge) are examined in detail.
**Chapter 5** describes the results from LDFE analyses for a T-bar penetrating into a single clay layer. This is to provide insight into the evolving soil failure mechanisms in both soft and stiff clays, and corresponding influence on penetration resistance profiles. Both centrifuge scale T-bar and field scale T-bar are considered, covering a practical range of $s_d/\gamma D_t = 1$~100. The influence of the T-bar surface roughness factor is also investigated. Based on the results, a systematic procedure is established for interpreting undrained shear strength from T-bar penetration data in a single clay layer.

**Chapter 6** reports the results from LDFE analyses for a T-bar penetrating into two-layer clays, such as stiff-soft and soft-stiff clay deposits. In order to interpret the layer boundaries and undrained shear strength of each individual layer, an interpretation framework is proposed considering the effects of layer thickness and soil strength ratio.

**Chapter 7** investigates the 3D effect (or end effect) on T-bar resistance factors and their corresponding soil flow mechanisms. A series of 3D small strain finite element analysis was conducted to study the effects of T-bar aspect ratio and roughness on T-bar resistance factors. A correction method accounting for the end effect is proposed as an adjustment factor to the existing or commonly used analytical plane strain solutions for an infinite long T-bar.

**Chapter 8** summarises the major research findings of this research. Suggestions for issues requiring further investigations are also included.

### 1.6 References

Chapter 1


Chapter 1

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Chapter 1


Introduction

Figure 1-1. Single, double and multilayered seabed profiles: (a) clay deposits (Gulf of Mexico and Sunda Shelf: after Handidjaja et al., 2004; Kostelnik et al., 2007; Menzies & Roper, 2008; Menzies & Lopez, 2011); (b) deposits with sand layers (Sunda Shelf, offshore India, Gulf of Mexico: after Purwana et al., 2009; Teh et al., 2009; Menzies & Lopez, 2011); (c) calcareous very soft to stiff clay (Gulf of Mexico: after Menzies & Lopez, 2011); (d) calcareous clay-sand-sandy silt (offshore Australia: after Erbrich, 2005; Watson & Humpheson, 2005)
Figure 1-2. Collection of in-situ testing tools (after Mayne, 2006)
Figure 1-3. Penetrometer test conduction platforms: (a) onshore (after United States Geological Survey’s website https://earthquake.usgs.gov/research/cpt/) (b) offshore (after Randolph & Gourvenec, 2011)
Figure 1-4. Cone, T-bar and ball penetrometers (after Robertson, 2012)
CHAPTER 2  SOIL FLOW MECHANISMS OF FULL-FLOW PENETROMETERS IN LAYERED CLAYS –
PIV ANALYSIS IN CENTRIFUGE TEST

ABSTRACT

Increasing consideration of full-flow penetrometers, including T-bar and ball, in offshore site investigation has been muddied by the lack of accurate interpretation framework for their resistance profile, especially in layered seabed sediments. The first step is to reveal the associated soil flow mechanisms. This chapter focuses on the soil flow mechanisms resulting from the continuous penetration of full-flow penetrometers accounting for the effect of soil layering. A series of centrifuge tests was conducted with the penetrometers penetrating through soft-stiff, stiff-soft, soft-stiff-soft, and stiff-soft-stiff clay profiles. Particle image velocimetry (PIV) allowed accurate resolution of the flow mechanism around a face of the T-bar and half-ball penetrated adjacent to a transparent window. For the T-bar, overall a symmetric rotational flow around the half section of the T-bar dominated the behaviour. A novel trapped cavity mechanism was revealed in stiff clay layer, with the evolution of the cavity being tracked. No trapped soil plug was detected at the base of the advancing T-bar regardless of penetration from stiff to soft layer or the reverse. For the ball, the key feature of soil flow is a combination of vertical flow, cavity expansion type flow and rotational flow. In stiff-soft clay deposit, punch-through occurred close to the layer interface that delayed the soil backflow process in the cavity above the ball and
deformed the layer boundary. A stiff clay plug was trapped at the base of the ball, which was forced into the softer underlying layer.

2.1 Introduction

Accurate characterisation of seabed sediments is crucial for the design of offshore foundations. Recent exploration of hydrocarbon products in deep water has placed more reliance on in-situ testing, such as vane shear testing and push-in penetrometer testing, due to the difficulty and high cost of obtaining high-quality soil samples for laboratory tests. Unlike vane shear test, push-in penetrometers (cone, T-bar, ball) generally provide continuous or near-continuous resistance profile and hence higher resolution of soil strength profiles and layer interfaces. This study focuses on T-bar and ball penetrometers (Figure 2-1; Randolph et al., 1998; Newson et al., 1999; Kelleher & Randolph, 2005; Peuchen et al., 2005) that are referred to ‘full-flow’ penetrometers as they allow soil to flow around the probes (except a small region where the shaft is connected to the ball). These penetrometers are increasingly being included in deep water soft soil characterisation program mainly due to relatively large projected area (5~10 times larger than cone), giving high resolution of penetration resistance profile, and negligible influence of overburden pressure to the total resistance. The undrained shear strength profile of fine-grained seabed sediments is therefore deduced according to

\[
    s_u = \frac{q_{\text{net}}}{N_p}
\]

(2-1)

where \(q_{\text{net}}\) is the tip net resistance, \(N_p\) is the penetrometer resistance factor (\(N_t\) for T-bar and \(N_b\) for ball) and \(s_u\) is the undrained shear strength of soil. Full-flow
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penetrometers have also been the common tools for characterising fine-grained sediments in both centrifuge and 1g laboratory experiments (Hossain et al., 2011).

The evolving soil failure mechanisms along the penetrometer penetration depths dictate the values of $N_p$. However, due to the lack of experimental investigation, an a priori assumption was made in terms of the failure mechanism and corresponding analytical solutions were derived. For the T-bar with large length-diameter ratio (i.e. 4~6.25), the plasticity solutions for pile and pipe for shallow embedment (Murff et al., 1989; Aubeny et al., 2005; Randolph & White, 2008) and for deep embedment (Randolph & Houlsby, 1984; Martin & Randolph, 2006) are considered. The assumed failure mechanisms are illustrated in Figure 2-2a~c. Analytical solutions for an embedded ball were reported by Randolph et al. (2000), with the assumed mechanism shown in Figure 2-3a.

Recently, the T-bar and ball penetration in clay is treated through large deformation finite element (LDFE) analysis. Pushing a plane strain cylinder bar continuously from the soil surface, White et al. (2010) found two mechanisms: shallow mechanism with an open cavity above the bar and deep flow-round mechanism around the fully embedded bar (Figure 2-2d); while Tho et al. (2011) identified a trapped cavity above a smooth bar up to a penetration of $10D_t$ in a relatively stiff clay ($s_d\gamma' D_t > 3$, where $\gamma'$ is the soil effective unit weight and $D_t$ is the bar diameter; Figure 2-2e). The trapped cavity led to a 12% lower $N_t$ compared to a flow-round mechanism, meaning the soil strength could be underestimated if the resistance factor for a flow-round mechanism is adopted. No investigation was carried out on T-bar penetration in layered clays.

For the ball penetrometer in single layer clay, LDFE analyses for a pre-embedded ball was performed by Zhou & Randolph (2009) and Zhou & Randolph (2011), with the
latter focused on the effect of the shaft-ball area ratio \( a = D_s^2/D_b^2 \); where \( D_s \) is the shaft diameter and \( D_b \) is the ball diameter). A combined mechanism of cavity expansion and localised backflow was exposed, as shown in Figure 2-3b. The resistance factor \( N_b \) for a ball with a shaft, as has also been investigated by Zhou et al. (2013, 2016), was shown to be lower than that of a spherical ball \( (a = 0) \).

Ball penetrometer (including a shaft) penetrating continuously from the surface through a single layer clay and layered soft-over-stiff and stiff-over-soft clay deposits has been investigated by Zhou et al. (2013, 2016). A shallow mechanism with the cavity above the penetrating ball open and a deep flow-round mechanism around the embedded ball along with sustained cavity above were reported. The effect of soil layering on the mobilised failure mechanisms, and critically on trapping of a stiff soil plug beneath the advancing ball, was shown to be significant.

Recently, half-model test against a window followed by particle image velocimetry (PIV) analyses has been a popular technique in centrifuge testing for revealing the true soil failure mechanisms (e.g. Hossain & Randolph, 2010; Hossain, 2014; Stanier et al., 2015). However, this technique has yet to be used for observing the soil failure mechanisms around penetrometers except Paniagua et al.’s (2013), Arshad et al.’s (2014) and Mo et al.’s (2015, 2017) investigation on the cone penetrometer in single layer and layered sands.

This chapter has therefore focused on revealing the soil failure mechanisms around full-flow penetrometers in layered clays. The soil movement was captured with penetrometers penetration through clays against a transparent window by a camera and then quantified through PIV analyses. The effect of soil layering on the mobilised soil flow mechanisms in soft-stiff, stiff-soft, soft-stiff-soft and stiff-soft-stiff deposits.
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has been addressed. The soil displacement path induced by penetrometers are tracked, and the layered effect is discussed through comparison of soil elements at various distances from the interface.

2.2 Centrifuge modelling

The experimental program was carried out at 50g in the beam centrifuge at the University of Western Australia (Randolph et al., 1991). The experimental set-up is shown in Figure 2-4. A purpose design PIV strongbox with a Plexiglas window was built to allow observation of the soil deformation through the window. The PIV box has an internal size of 337 mm (length) × 100 mm (width) × 299 mm (depth), and it was fitted tightly at one side of a beam standard strongbox (see Figure 2-4a and b). The window was faced to the opposite side of the beam standard strongbox where a camera was mounted for capturing images.

2.2.1 Model penetrometers

Two model penetrometers machined from duraluminum were used (Figure 2-5a). The surfaces of the penetrometers were relatively smooth as they were anodised and not sand blasted during fabrication. The model T-bar consisted of a cylindrical bar of diameter \(D_t\) 15 mm and length 100 mm (0.75 m × 5 m at 50g) with a 15 mm cylindrical shaft connected to the middle of the bar, which allowed to push one face of the bar adjacent to the strongbox window (and spanning across the PIV strongbox; see Figure 2-4b) vertically without any other support, permitting the soil deformation to be captured by the camera. To achieve proper sealing against the strongbox front window and back plate, predominantly preventing soil ingress between the foundation and the window, two O-ring strips were fitted into machined grooves along the two opposite edges of the bar. An O-ring is a loop of elastomer with a round cross-section,
which is designed to be seated in a groove and compressed during assembly between two or more parts, creating a seal at the interface.

For the ball penetrometer, by contrast, a half model of 20 mm diameter ($D_b$, 1 m at 50g) attached with a 10 mm half cylindrical shaft ($D_s$) facilitated penetration adjacent to the window. A 7 mm thick board was connected to the back of the shaft at 30 mm ($1.5D_b$) away from the half sphere. This was to strengthen the shaft, minimising potential backward bending while allowing the soil to flow back around and above the penetrating ball. Again, an O-ring was fitted along the periphery of the flat face of the half ball, which would be compressed against the window during penetration.

Figure 2-5b shows schematic diagram of the two penetrometers penetration in layered clay, with $d$ referring to the distance from the tip of the probes to the surface of the soil and $t$ to the distance from the tip of the penetrometer to the underlying soil layer interface. The layer shaded with light grey is to represent the stiff clay layer, and this is consistent with all the mechanism figures in this chapter.

The tests were performed at least 80 mm apart and at a distance of 70~85 mm from the side boundary (Figure 2-4b). The final penetration depth was around 220 mm, giving a 50 mm (3.33$D_t$ for the T-bar and 2.5 $D_b$ for the ball penetrometer) clearance from the bottom boundary. These distances allowed for precluding boundary influence and in between interference, which was confirmed by the measured penetration resistance profiles (with no significant increase when approaching its final penetration depth) and soil failure mechanisms. Further, from an extensive investigation, Ullah et al. (2014) reported that, in clay, a distance of 0.7~1.0 dimeter of the penetrating object is sufficient for avoiding the influence of the boundary.
Soil flow mechanisms of full-flow penetrometers in layered clays – PIV analysis in centrifuge test

The T-bar and ball were penetrated at \( v = 0.2 \text{ mm/s} \) and \( 0.15 \text{ mm/s} \) respectively, ensuring undrained conditions (Finnie & Randolph, 1994) with a dimensionless velocity \( V = \frac{vD}{c_v} = 37 \) (\( c_v \approx 2.6 \text{ m}^2/\text{year} \)) for both penetrometers, while allowing adequate picture taking frequency for PIV analysis.

2.2.2 Sample preparation and soil strength characterisation

Kaolin clay slurry was deaired and pre-consolidated under two final pressures of 100 and 400 kPa to produce soft and stiff clay samples respectively. According to the predesigned thicknesses of the layers (listed in Table 2-1), rectangular blocks were sliced from the pre-consolidated samples and slid in the PIV strongbox. To add texture to the white kaolin clay for PIV analysis and to track the deformation of one layer into another layer, green and black flock powders were sprinkled respectively on the stiff and soft clay layer sides facing the window (Dingle et al., 2008; Hossain & Randolph, 2010).

Four boxes were prepared varying the strength profile as soft-stiff, stiff-soft, soft-stiff-soft, stiff-soft-stiff sediments (Table 2-1). In each box, a miniature T-bar (5 mm × 20 mm) and ball (9 mm) penetrometer tests were carried out in-flight, at the penetration rate of 1 and 1.25 mm/s respectively, which was sufficiently fast to ensure undrained conditions in kaolin clay (Finnie & Randolph, 1994). The resistance profiles in layered sediments will be discussed in the next section. Moisture content tests were conducted after all penetration tests. The effective unit weights (\( \gamma' \)) of the soft and stiff layers were \(~7.0 \text{ kN/m}^3\) and \(~7.4 \text{ kN/m}^3\) respectively.

2.2.3 Image capture and PIV analysis

A Prosilica GC2450C digital still camera was set up in continuous shooting mode to capture images of the penetrometer and the surrounding soil throughout the tests. The
5 megapixel digital images were taken at every 0.05 mm penetration depth increment for both T-bar and ball penetrometers. A 50-mm grid consisting of 24 control points (black dot on a white background) was installed within the Plexiglas window. A centroiding technique, based on these control points coordinates, was implemented to calibrate the image space to the object space. In this way, any subsequent image-space displacement of the control markers incurred by the camera movement was evaluated. Full details of the PIV and photogrammetric analysis can be found in White et al. (2003) and Stanier et al. (2015).

2.3 Results and discussion

Throughout this section, due to the symmetry of the penetrometers, soil failure mechanisms figures are presented using soil displacement vectors on the left and displacement contours on the right. The displacement fields were resulted from a penetration interval of $\Delta d = 2.2$ mm, which is equivalent to the displacement interval during 45 continuous images, with each PIV calculation consisting of 10 images at an interval of 5. This displacement interval was selected through try and error. Similar level of displacement interval has been selected in Ni et al. (2010). The vectors were scaled up by 4 times to provide a better resolution. The displacement contours were obtained normalising the soil displacement by the penetration interval, ranging from 0.1 to 0.9. The soil displacement beyond the contour of 0.1, corresponding to the total soil displacement of 0.22 mm, was taken as ‘negligible movement’, giving a quantitative measurement of the soil displaced zone. Prior to the flow mechanism results from PIV T-bar and ball, the results of resistance profiles from miniature T-bar and ball testing are presented.
2.3.1 Resistance profiles

Figure 2-6a presents the resistance profiles from the miniature T-bar test in B1~4. The profiles of resistance, \( q = F/A \) (\( F \) is measured tip resistance and \( A \) is the projection area), are plotted as a function of penetration depth, \( d \), in prototype. The layer interface information is also marked with the dot dashed line in the corresponding colour. It can be seen that, the resistance profiles decrease abruptly, just at or above the stiff-soft layering interface followed by a relatively gradually decreases in the soft layer. In contrast, for soft-stiff layering, the sudden increase in resistance occurs from a distance above the interface, but it increases relatively sharply in the stiff layer. This transition zone corresponds to the process with gradual involvement of the underlying soil and departing the soil above interface. The process of T-bar and ball passing through the layer interfaces will be illustrated extensively with PIV results.

For B1 in the soft 1st layer, the thickness of \( h_1/D_t = 12.4 \) is sufficient to mobilise the full (ultimate) penetration resistance of that layer. However, the thickness of \( h_1/D_t = 4.6 \) (B3) is apparently not sufficient. As such, the ultimate resistance in the top layer of B3 before sensing the stiff clay is lower that of of B1. Therefore, the most reliable undrained shear strength for soft clay layer in this test is based on the ultimate pressure for 1st layer of B1, i.e. 78.8 kPa, which leads to \( s_u = 7.5 \) when normalized resistance factor taken as 10.5 (Randolph & Houlsby, 1984).

In the stiff 1st layer, the thickness of \( h_1/D_t = 12.4 \) (B2) is apparently not sufficient to mobilise the full (ultimate) resistance of that layer, as well as the interbedded stiff layer in B3. The full resistance may be significantly higher as can be seen from the resistance in the stiff 2nd layer (B1) and stiff 3rd layer (B4). As such, the undrained shear strength of stiff clay is further determined with ball resistance data in the above two layers (Figure 2-6b). Based on this larger database, the average resistance is found
at 314.5 kPa, with average undrained shear strength equalling to 29.1 kPa when using normalized resistance for T-bar (10.5, (Randolph & Houlsby, 1984)) and ball (11.2 when shaft-ball diameter ratio = 0.5, (Zhou & Randolph, 2011)) adopted respectively.

2.3.2 Soil flow mechanisms: T-bar penetrometer

2.3.2.1 T-bar penetration in soft-stiff clay

Figure 2-7 depicts the soil flow mechanisms in a two-layer soft-stiff clay deposit ($h_1/D_t = 12.4, s_{u2}/s_{u1} = 3.88; B1, TB1; Table 2-1$). The transition from a shallow mechanism with an open cavity to a stabilised deep full flow-round mechanism (Figure 2-7a–c) is observed in the top soft clay layer. At $d/D_t = 2$ (Figure 2-7a), the soil flow is directed mostly downward, outward, followed by upward towards the surface, resulting in an open cavity above the advancing T-bar. The lateral soil displaced zone extends to $x/D_t = 1.8$. With the increase of penetration depth and hence overburden pressure ($d/D_t = 5.9; Figure 2-7b$), the upward flow direction is gradually changed to flow above the T-bar, leading to the closure of the cavity wall and the mobilisation of a flow around mechanism. At $d/D_t = 9.8$ (Figure 2-7c), similar to Figure 2-7b, a nice full flow mechanism is captured.

When the T-bar approximates to the soft-stiff layer interface by $t/D_t = 0.4$ ($d/D_t = 11.8; Figure 2-7d$), a squeezing out mechanism dominates the behaviour with soil flow being restricted by the underlying stiff clay layer with $s_{u2}/s_{u1} = 3.88$, indicating negligible deformation in the lower layer and of the layer interface. A clean T-bar with no trapped soil plug is therefore enters into the stiff clay layer. The soil flow gradually becomes localised, with the soft clay above the advancing T-bar being pushed up by the backfilled stiff clay ($d/D_t = 13.9; Figure 2-7e$).
More insight of the mechanism in the top soft layer can be found tracking the trajectory of some materials. Five soil elements (M1~M5) at ~4.4\(D_t\) above the layer interface and five elements (N1~N5) at just ~0.3\(D_t\) above the interface were tracked, as shown in Figure 2-8. The lateral distances between the elements and the centreline of the T-bar are \(x/D_t = 0.6, 0.7, 1, 2, 3\). The resultant of horizontal displacement (\(\Delta x\)) and vertical displacement (\(\Delta y\)) of each element due to the incremental penetration of the T-bar from \(y_1/D_t = 3\) above to \(y_2/D_t = 2.7\) below the original level of the element provided the trajectory of the element.

Figure 2-9a shows the trajectories of soil elements M1~M5 indicating generally a rotational flow around path (e.g. for M1 labelled as O → A → B → C → D). As expected, larger soil movement is associated with closer elements. The downward movement becomes shallower and lateral outward movement shorter as element distance \(x/D_t\) increases. As the T-bar approaches and leaves element M1, the normalised incremental horizontal (\(\Delta x/D_t\)) and vertical (\(\Delta y/D_t\)) displacements are plotted in Figure 2-9b as a function of \(y/D_t\) (‘-ve’ means above the original level of the element). Both \(\Delta x/D_t\) and \(\Delta y/D_t\) change sharply, increase followed by decrease, only for \(y/D_t = \pm 1\), which is termed as ‘active zone’. For \(\Delta y/D_t\), the peak value appears at \(y/D_t = 0\) (at B in Figure 2-9a) i.e. when the T-bar tip is at the element original level, while for \(\Delta x/D_t\), the peak value appears at \(y/D_t = 0.5\) (at C in Figure 2-9a) i.e. when the center of T-bar reaches the original level. Finally, the element M1 eventually rests at 0.07\(D_t\) above and 0.13\(D_t\) away from its original location after being displaced by the T-bar penetration.

The gradients of \(\Delta y/D_t\) and \(\Delta x/D_t\) profiles in Figure 2-9b suggest constantly changing velocity of element M1 with T-bar penetration, which can be explained further plotting.
the element resultant velocity relative to the T-bar penetration velocity ($v_{\text{soil particle}}/v_{\text{T-bar}}$) as well as the relative position of the soil element (square) and T-bar (patched circle) at critical positions in Figure 2-9c. As the T-bar approaches and leaves the element, the normalised velocity increases from 0.025 to 0.3 to 0.4 (A to B to C), forms a peak at around C, decreases to 0.25 (at D), followed by drops drastically to 0.025 i.e. the element becomes inactive.

In Figure 2-9d, the soil particles trajectories at $x/D_t = 0.6, 0.7, 1$ have been compared directly with the classical flow-round mechanism form Martin & Randolph (2006) for a relatively smooth (roughness factor of 0.3) T-bar used in this study. In contrast to the tracked trajectories from the centrifuge test, (i) the classical mechanism does not involve any downward movement prior to flow back, which is because analytical solutions assume soil as a rigid plastic material neglecting the elastic downward-outward compression as the T-bar approaches the soil particle; (ii) the upward movement in the classical mechanism is 3~8 times higher, which might due to the fact that analytical solutions do not consider confined pressure; (iii) at the end of movement the particles come to zero lateral location line ($\Delta x/D_t = 0$) i.e. completes the loop. Nevertheless, the maximum lateral displacements for $x/D_t = 0.6, 0.7$ are very similar.

The tracked trajectories of five elements near the soil layer interface (N1~N5 in Figure 2-8) are shown in Figure 2-10. The effect of squeezing can be quantified comparing the various features of the trajectories in Figure 2-10 (with the influence of the underlying stiff layer; $s_{u2}/s_{u1} = 3.88$) with those in Figure 2-9 (without the influence of the underlying layer). As the T-bar approaches and leaves the elements, while trends of the trajectories are very similar, (i) the maximum downward movement of e.g. element N1 (at B in Figure 2-10a) becomes around half of that of element M1 (at B in
Figure 2-9a) being restricted by the underlying stiff layer; (ii) the maximum lateral displacement of element N1 in Figure 2-10b is three times of the vertical displacement whereas that of M1 in Figure 2-9b is about same as vertical displacement; and finally (iii) element N1 travels back to around its original position but element M1 does not.

2.3.2.2 T-bar penetration in stiff-soft clay

Figure 2-11 displays the soil flow mechanisms in a two-layer stiff-soft clay deposit \((h_2/D_t = 12.4, \frac{s_{u2}}{s_{u1}} = 0.26; \text{B2, TB2; Table 2-1})\). For identical penetration depths up to \(-9.7D_t\), the flow mechanisms in single layer stiff clay \((s_{u1}/\gamma'_{1}D_t = 5.24; \text{without the influence of the underlying layer})\) illustrated in Figure 2-11a~c are principally similar to those in soft clay \((s_{u1}/\gamma'_{1}D_t = 1.42)\) shown in Figure 2-7a~c. Only exception, and interestingly, an open cavity is trapped above the advancing T-bar in stiff clay \((s_{u1}/\gamma'_{1}D_t = 5.24)\) for \(d/D_t \geq 4.7\), hindering the mobilisation of full flow-round mechanism. This phenomenon was also exposed by Tho et al. (2011) through numerical analysis of a pipe penetration in uniform single layer clay with \(s_u/\gamma' D_t > 3\), and the trapped cavity was termed as ‘deep cavity mechanism’ (see Figure 2-2e).

With the progress of penetration, the trapped cavity is become narrower by the continual backfilled soil. When the T-bar tip is at the layer interface \((d/D_t = 12.4; \text{Figure 2-11d})\), with the trapped cavity still existing, a localised flow-round mechanism mobilises some of the soil from underlying soft layer, leading to downward deformation of the layer boundary. At \(d/D_t = 14.0\) (Figure 2-11e) with the T-bar in the soft clay layer, a column of stiff clay forms around the trapped cavity above the advancing T-bar. Finally, the T-bar separates from the stiff clay layer and the trapped cavity disappears at \(d/D_t = 14.7\) (Figure 2-11f). From Figure 2-11e and f, no trapped
stiff soil plug can be observed at the base of the advancing relatively smooth T-bar when pushed in the soft layer.

In offshore in-situ site investigation, the observed trapped cavity will be filled with water, same as this study, and the corresponding influence will be a reduction of resistance or bearing capacity factor compared to the full flow-round mechanism as is considered in conventionally used plasticity solutions (discussed in introduction). To quantify the influence of the trapped cavity on the bearing capacity factors, a series of limit analysis were conducted for the mechanisms observed from Figure 2-11a~e using the software Optum G2 (Krabbenhøft et al., 2014). The results are shown in Figure 2-12. For comparison, Figure 2-12 also includes (i) the bearing capacity factors in top layer clay (for $d/D_t \leq 10$) using the approach proposed by White et al. (2010), which considered an open cavity at shallow depths (similar to Figure 2-11a) but a full flow-round mechanism at deeper depths without any trapped cavity; and (ii) the bearing capacity factors above and below the interface (for $d/D_t = 10$~14) for a pre-embedded T-bar with mobilised flow-round mechanism from limit analyses, accounting for the interface and bottom soft soil layer but without the trapped cavity and interface deformation (see the insert in Figure 2-12). It is apparent that the observed soil failure mechanisms-based bearing capacity factors are 10~18% lower than the classical mechanism-based plasticity solutions. Tho et al. (2011) quantified this reduction as 12%. Consequently, in the field, the interpreted shear strength would be 10~18% underestimated if trapped cavity is not considered.

Similar to soft-stiff clay deposit, five elements M1~M5 were tracked 3.4$D_t$ away from the layer interface, and the trajectories are shown in Figure 2-13. By comparing Figure 2-13 with Figure 2-9 for e.g. element M1, the vertical downward displacement is
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significantly higher whereas the vertical upward displacement is markedly lower. For instance, for element M1, the downward displacement in Figure 2-13b accumulated in O → A → B is almost twice of the upward displacement during B → C → D for stiff clay, whereas in soft clay (Figure 2-9b) the downward displacement is only half of upward displacement. This is mainly because the trapped cavity formed in the stiff layer hinders the soil backflow so the downward displacement is strengthened and backward movement is reduced.

Figure 2-14 shows the trajectories of soil elements (N1~N5) near the stiff-soft interface (t = ~0.1Dt). The relative trend between Figure 2-13 and Figure 2-14 (with and without the underlying layer influence) is reverse of that between Figure 2-9 and Figure 2-10 as just discussed in the previous section. For elements N1~N3, both lateral and vertical displacements are higher. This is because for soft-stiff clay the underlying layer restricted the soil downward flow and for stiff-soft clay the soil flow is being attracted by the underlying layer.

2.3.2.3 T-bar penetration in soft-stiff-soft and stiff-soft-stiff clays

The influence of three-layer soft-stiff-soft layering on the evolution of the T-bar penetration mechanisms is illustrated in Figure 2-15 (h1/Dt = 4.6, h2/Dt = 5.3, su2/su1 = 3.88, su3/su2 = 0.26; B3, TB3; Table 2-1). The squeezing mechanism in Figure 2-15a is consistent to what is presented in Figure 2-7d for soft-stiff clay apart from earlier backflow owing to the earlier influence of the 2nd (stiff) layer caused by the thinner 1st layer (h1/Dt = 4.6 in Figure 2-15a vs. 12.4 in Figure 2-7d). The mechanisms in Figure 2-15b and c are very similar to what presented in Figure 2-11d and e for stiff-soft clay apart from the absence of the trapped cavity.
To examine the effect of three-layer stiff-soft-stiff layering, the corresponding mobilised soil flow mechanisms are shown in Figure 2-16 ($h_1/D_t = 5.2$, $h_2/D_t = 5$, $s_{u2}/s_{u1} = 0.26$, $s_{u3}/s_{u2} = 3.88$; B4, TB4; Table 2-1). For the 1st and 2nd stiff-soft layering ($s_{u2}/s_{u1} = 0.26$; $s_{u1}/\gamma'_1 D_t = 5.24$), the thin top stiff clay layer ($h_1/D_t = 5.2$) allows for earlier attraction of the soft layer from nearly the beginning of the T-bar penetration. This delays the soil backflow and the soil deformation is directed predominantly vertically downward to the lower soft layer, leading to (i) an open cavity above the T-bar throughout the penetration in the top layer, and (ii) deformation of the stiff-soft layer interface. The behaviour of T-bar in the top stiff layer clay can be compared between TB4 and TB2 (Figure 2-16 vs. Figure 2-11). The mechanisms in Figure 2-11a-c ($d/D_t = 1.5$~9.7) in stiff-soft clay with a thicker top layer ($h_1/D_t = 12.4$) can be taken as more similar to the mechanisms in single layer stiff clay. For Figure 2-11b and Figure 2-16b, although $s_{u1}/\gamma'_1 D_t$ of 5.24 and penetration depth $d/D_t$ (i.e. 4.7) are identical, interestingly, no trapped cavity is formed in Figure 2-16b as like Figure 2-11b, rather the cavity remains open in the top layer in Figure 2-16b. This reflects the earlier influence of the underlying soft layer. The soil starts to flow back and covers the top of the T-bar immediately after penetrating in the soft layer (Figure 2-16c). This is consistent with the observation of Hossain & Randolph (2010) for a spudcan penetration in stiff-soft clay deposits. The mechanisms in Figure 2-16c~e are similar to Figure 2-11d~f apart from the absence of the trapped cavity above the T-bar. For the 2nd to 3rd soft-stiff layering ($s_{u3}/s_{u2} = 3.88$), the squeezing mechanisms illustrated in Figure 2-16f and g are consistent with those in Figure 2-7d and e.

The influence of the mechanisms observed in Figure 2-16 on the corresponding T-bar penetration resistance profile (TB4) is highlighted in Figure 2-17. The depths of Figure 2-16a to g are marked in Figure 2-17. For the thin stiff layer, due to early sensing the
Influence of the 2\textsuperscript{nd} (soft) layer and presence of an open cavity above the penetrating T-bar (Figure 2-16a and b), the penetration resistance profile does attain to the ultimate (maximum) resistance of the layer followed by a steady profile as like the profile in a single layer clay. Instead, it increases followed by sharply decreases. The influence of the mechanisms in Figure 2-16c–e, with the proportion of mobilised soil flow in the soft soil increases and that in the top layer stiff soil decreases, is the continual decrease in resistance profile in the 2\textsuperscript{nd} (soft) layer. At Figure 2-16e, the flow-round mechanism occurs fully in the soft layer (without the influence of the top and bottom layers). As such, the penetration resistance profile approaches to the ultimate resistance of the soft layer. Figure 2-16f and g show that the proportion of soil flow increases in the bottom stiff layer and decreases in the soft layer, leading to a continual increase in penetration resistance. In summary, the soil layering and mobilised soil failure mechanisms have significant influence in the penetration resistance profile, which should be taken into account in the interpretation of shear strength from the measured penetration resistance profile. For instance, due to the thin top layer ($h_1/D_t = 5.2$), the measured maximum resistance in the top layer is significantly lower than that at $d/D_t = 13.5$ in the bottom layer. Using a constant bearing capacity factor would lead to significantly lower undrained shear strength of the top layer.

In summary, eight interesting features of soil flow mechanisms that are expected to have significant influence on the bearing capacity factor and hence on the interpreted shear strength include; (i) overall a symmetric rotational flow around the half section of the T-bar is the key feature; (ii) open cavity and subsequent backflow and replenish the cavity in soft clay regardless of $h_1/D_t$ with soft enough sediments ($h_1/D_t = 12.4$ and 4.6, $s_{u2}/s_{u1} = 3.88$, $s_{u1}/\gamma'_{1}D_t = 1.42$, 1\textsuperscript{st} layer in TB1 and TB3); (iii) open cavity and subsequent backflow above the T-bar maintaining a trapped cavity in deep stiff clay
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with $h_1/D_t = 12.4$, $s_{u2}/s_{u1} = 0.26$ ($s_{u1}/y' D_t = 5.24$, 1st layer in TB2); (iv) open cavity with delayed backflow up to the stiff-soft interface in a thin stiff clay layer with $h_1/D_t = 5.2$, $s_{u2}/s_{u1} = 0.26$ ($s_{u1}/y' D_t = 5.24$, 1st layer in TB4); (v) squeezing close to soft-stiff interface with negligible deformation of the interface; (vi) deformation of stiff-stiff interface with partial flow into the underlying soft layer; (vii) no trapping of soil plug at the base of the advancing T-bar regardless of penetration from stiff to soft layer or the reverse; (viii) stiff or soft soil column above the T-bar while penetrating from stiff to soft or soft to stiff layer, respectively, and finally detaching from the upper layer.

By comparing with the conventional mechanisms (see Figure 2-2) used for developing plasticity solutions, the full flow-round mechanism in Figure 2-2a and flow-round mechanism cut off by the open cavity in Figure 2-2c are consistent with the observed mechanisms in thick soft clay layer illustrated in Figure 2-7c and Figure 2-7a & Figure 2-11a, respectively. The other observed evolving mechanisms in thick stiff clay layer and in layered soils are different, urging the necessity of improving bearing capacity factors accounting for the mobilised failure mechanisms for more accurate interpretation of undrained shear strength from in situ T-bar test data.

2.3.3 Soil flow mechanisms: ball penetrometer

The mechanisms for the axisymmetric ball penetrometer will be discussed in this section by comparing with those presented in Figure 2-7~Figure 2-16 for the plane strain T-bar.

2.3.3.1 Ball penetration in soft-stiff clay

Figure 2-18 displays the soil flow mechanisms in a two-layer soft-stiff clay deposit ($h_1/D_b = 9.3$, $s_{u2}/s_{u1} = 3.88$; B1, Ball1; Table 2-1). A general shallow shear failure leads to a cavity above the ball. This is followed by soil flow around the ball (Figure
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2-18a–b). By comparing Figure 2-18a–c with Figure 2-7a–c, for the T-bar, the deep flow-round mechanism occurs around nearly the whole (half) T-bar, whereas for the ball, that takes place around the upper section of the (half) ball (as is evidenced by the zoomed figure in Figure 2-18) and a classical cavity expansion type failure occurs around the lower section. To further explore the mechanism around the ball, streamlines are fitted through flow directions of incremental movement of Figure 2-18c, and presented in the left part of Figure 2-19. Based on the streamlines, the displacement field can be divided into three zones: (i) A-zone in the extension line of the shaft; (ii) B-zone between A-zone boundary and H-line, which is the horizontal level 0.3\(D_b\) above the ball tip; (iii) C-zone between H-line and the shaft of the ball. The soil in A-zone directs predominantly vertically downward, and that in C-zone, flows around the ball. The soil in B-zone flows downward followed by laterally outward i.e. a cavity expansion type failure takes place. The open cavity is closed by the backfilled soil in Figure 2-18c.

Near the soft-stiff interface, squeezing mechanism mobilises in Figure 2-18d. Finally, the ball penetrates in the lower stiff layer with no trapping of soft soil plug underneath as is the case for the T-bar.

Five soil elements (M1–M5) at ~2.4\(D_b\) above the layer interface and five elements (N1–N5) at just ~0.1\(D_b\) above the interface are tracked during the ball penetration from ~2.5\(D_b\) above the element level to ~1.7\(D_b\) below the level. The lateral distances between the elements and the centreline of the T-bar are \(x/D_b = 0.6, 0.7, 1, 2, 3\). The trajectories of M1–M5 in \(\Delta x/D_b, \Delta y/D_b\) space are shown in Figure 2-20a, while the values of \(\Delta x/D_b\) and \(\Delta y/D_b\) of M1, as a function of \(y/D_b\), are plotted in Figure 2-20b. As \(x/D_b\) increases, i.e. from M1 to M2, both vertical and lateral displacements reduce.
For element M1, it gradually moves downward and outward ($O \rightarrow A \rightarrow B$) until the H-line reaches the same level with the element ($y = 0.3D_b$), which is consistent with the flow pattern observed in A-zone and B-zone (Figure 2-19). When the element passes the H-line and enters in C-zone, it flows back ($B \rightarrow C \rightarrow D$) until the element is about the same height with the shoulder of the ball ($y = 0.8D_b$). The backflow displacement is much lower compared to downward and outward displacement ($O \rightarrow B$). From Figure 2-20b, the vertical displacement $\Delta y/D_b$ peaks at B and lateral displacement $\Delta x/D_b$ at C, with the peak value of $\Delta y/D_b$ being nearly half of the peak value of $\Delta x/D_b$.

The trend of the trajectories of elements N1~N5, as illustrated in Figure 2-21, are similar to those of M1~M5. The values of $\Delta x/D_b$ are also consistent, but the values $\Delta y/D_b$ are significantly lower owing to the influence of squeezing.

2.3.3.2 Ball penetration in stiff-soft clay

Figure 2-22 shows the soil flow mechanisms in a two-layer stiff-soft clay deposit ($h_1/D_b = 9.3$, $s_{u2}/s_{u1} = 0.26$; B2, Ball2; Table 2-1). The flow mechanism in Figure 2-22c for stiff clay ($s_{u1}/\gamma'_{1}D_b = 3.93$) is very similar to that in Figure 2-18c for soft clay ($s_{u1}/\gamma'_{1}D_b = 1.07$) apart from the attraction of the lower soft layer and hence deeper deformation in Figure 2-22c compared to restriction by the lower stiff layer in Figure 2-18c. By comparing the mechanisms in stiff clay presented in Figure 2-22c for the ball ($s_{u1}/\gamma'_{1}D_b = 3.93$) and in Figure 2-11c for the T-bar ($s_{u1}/\gamma'_{1}D_t = 5.24$); no trapped cavity can be observed above the advancing ball.

Close to the stiff-soft layer interface (just above and below; Figure 2-22b~d), a ‘punch-through’ type mechanism occurs beneath the axisymmetric ball, with the soil deformation being directed predominantly vertically downward to the lower layer and
in the lower layer. Here the term ‘punch-through’ only indicates the similarities of the observed mechanism with that has been revealed with spudcan over stiff-soft interface (Hossain & Randolph, 2010), because displacement controlled penetrometer test does not involve punch-through risk. This leads to deformation of the layer interface, trapping of a stiff clay plug at the base of the ball, which is forced into the softer underlying layer. The small plug height reflects the fact that a smooth ball has used in testing. According the results from large deformation analyses reported by Zhou et al. (2013), the plug height increases with the ball base roughness. For the plane strain T-bar, in contrast, a rotational failure occurs around the T-bar through both layers, resulting in no trapping of a stiff clay plug at the base of the T-bar (see Figure 2-11d and e).

Similar to Figure 2-20 and Figure 2-21, ten elements are tracked, with M1~M5 being at ~3.1$D_b$ and N1~N5 at ~0.2$D_b$ from the stiff-soft interface, and the trajectories are shown in Figure 2-23 and Figure 2-24 respectively. By comparing the trajectories in stiff clay (Figure 2-23) with those in soft clay (Figure 2-20), the downward and outward displacements are markedly greater in stiff clay. The peak displacements at Point B, $\Delta x/D_b$ and $\Delta y/D_b$, are nearly double in stiff clay (see Figure 2-23b and Figure 2-20b). These larger displacements might be caused by two reasons. First, in the soft clay with lower $s_u'/\gamma' D_b$, the soil tends to flow back early, leading to relatively less outward displacement. This phenomenon can be identified in the flow filed at Point B of Figure 2-20a and Figure 2-23a, where the average vector of vectors falling within B-zone / C-zone are compared, and the average zone is 2$D_b$ from the ball centre. The average vector is normalised by the ball penetration interval (2.2 mm in model scale) and expressed with its $x$- and $y$- component. Clearly, at these penetration depths, in the soft clay, the soil backflow tendency is more prominent with larger average vector in
C-zone listed in Figure 2-20a compared to Figure 2-23a, whereas in the stiff clay, the soil outward flow tendency is more prominent. Second, the influence of the underlying layer. The influence zone of the underlying layer, while the ball penetrating in the top layer, in stiff-soft clay is significantly larger than that in soft-stiff clay. Zhou et al. (2013) reported this zone as ~4$D_b$ above the layer interface for stiff-soft clay and only ~0.25$D_b$ for soft-stiff clay. In Figure 2-23 the soil elements at ~3.1$D_b$ (i.e. within the underlying layer influence zone) whereas in Figure 2-20 the elements are at ~2.4$D_b$ (i.e. far beyond the underlying layer influence zone). The attraction of a soft layer facilitates the soil to deform more vertically downward and laterally outward.

For soft-stiff clay near the interface (Figure 2-21), $\Delta x/D_b$ has dominated the displacement owing to squeezing. By contrast, for stiff-soft clay close to the interface (Figure 2-24), $\Delta y/D_b$ dictates the displacement due to the influence of punch-through failure or attraction from the bottom soft layer. For element N1, the peak values of $\Delta x/D_b$ and $\Delta y/D_b$ are very similar.

### 2.3.3.3 Ball penetration in soft-stiff-soft and stiff-soft-stiff clays

The influence of three-layer soft-stiff-soft layering on the evolution of the ball penetration mechanisms is illustrated in Figure 2-25 ($h_1/D_b = 3.4$, $h_2/D_b = 4.0$, $s_{u2}/s_{u1} = 3.88$, $s_{u3}/s_{u2} = 0.26$; B3, Ball3; Table 2-1). For the 1$^{st}$-2$^{nd}$ layers, the mechanisms in Figure 2-25a and b are consistent to the patterns illustrated in Figure 2-18d and e for soft-stiff clay. For the 2$^{nd}$-3$^{rd}$ layers, the punch-through pattern and trapping of a stiff soil plug at the base of the ball in Figure 2-25c and d are similar to those in Figure 2-22c and d. The stiff soil column above the ball exist up to $d/D_b = 9.8$ i.e. 2.4$D_b$ penetration in the bottom soft layer.
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To examine the effect of three-layer stiff-soft-stiff layering, the corresponding mobilised soil flow mechanisms are shown in Figure 2-26 ($h_1/D_b = 3.9$, $h_2/D_b = 3.7$, $s_{u2}/s_{u1} = 0.26$, $s_{u3}/s_{u2} = 3.88$; B4, Ball4; Table 2-1). Punch-through dominates the behaviour from the onset of the ball penetration due to the earlier attraction of the 2nd (soft) layer, compared to Figure 2-22, caused by the thinner 1st layer ($h_1/D_t = 3.9$ for Figure 2-26 vs. 9.3 for Figure 2-22). This delays soil backflow, resulting in a deeper cavity depth. The soil (in C-zone) starts to flow back above the ball after the ball penetrating in the 2nd (soft) layer, as shown by the zoomed inset in Figure 2-26d. With the progress of penetration in the 2nd (soft) layer (Figure 2-26d–f), the cavity replenishes, the trapped soil plug at the base of the ball remains, and a stiff soil column forms above the ball. The connection between the ball and the 1st (stiff) layer through the stiff soil column has separated completely at $d/D_b = 5.5$ (Figure 2-26f) i.e. after 1.6$D_b$ penetration in the 2nd layer. The apparent decreasing of the trapped plug height may be caused by the pushing back from the window rather than actual diminishing (Figure 2-26g), as evidenced by Ullah et al. (2017) through comparison of mechanisms associated with a conical and flat footing and post-test dissected soil plugs. The squeezing, negligible deformation of the soft-stiff layer interface and post-squeezing penetration mechanism in the stiff layer (Figure 2-26g–h), all are similar to Figure 2-18d and e for soft-stiff clay deposit.

In summary, nine interesting features of soil flow mechanisms that are expected to have significant influence on the bearing capacity factor and hence on the interpreted shear strength include; (i) overall a combination of vertical flow in A-zone, a cavity expansion type flow in B-zone and rotational flow around the top 0.7$D_b$ section of the ball in C-zone is the key feature; (ii) open cavity and subsequent backflow and replenish the cavity in soft clay regardless of $h_1/D_b$ ($h_1/D_b = 9.3$ and 3.4, $s_{u2}/s_{u1} = 3.88$,}
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$s_u/s_u' D_b = 1.07$, $1^{st}$ layer in Ball1 and Ball3); (iii) open cavity and subsequent backflow above the ball in deep stiff clay with $h_1/D_b = 9.3$, $s_{u2}/s_{u1} = 0.26$ ($s_{u1}/s_{u2} = 0.26$, $1^{st}$ layer in Ball2); (iv) open cavity with delayed backflow up to the stiff-soft interface in a thin stiff clay layer with $h_1/D_b = 3.9$, $s_{u2}/s_{u1} = 0.26$ ($s_{u1}/s_{u2} = 0.26$, $1^{st}$ layer in Ball4); (v) squeezing close to soft-stiff interface with negligible deformation of the interface; (vi) no trapping of soft soil plug at the base of the advancing ball penetrating from soft to stiff layer; (vii) punch-through failure along with deformation of stiff-soft interface; (viii) stiff soil plug at the base of the advancing ball penetrating from stiff to soft layer; (ix) stiff or soft soil column above the ball while penetrating from stiff to soft or the reverse, respectively, and finally detaching from the upper layer.

By comparing with the existing mechanisms (see Figure 2-3), neither the full flow-round mechanism (Figure 2-3a) assumed by Randolph et al. (2000) nor the combined mechanism (Figure 2-3b) proposed by Zhou & Randolph (2011) is observed. Rather, a combination of vertical flow, a cavity expansion type flow, and rotational flow around the top part of the ball dominates the behaviour along with the other effects of soil layering. In particular, the effect of the trapped soil plug may be significant as presented by Zhou et al. (2013) from large deformation finite element analyses as a function of ball base roughness.

2.4 Concluding remarks

This chapter presents the soil flow mechanisms a series of centrifuges tests on T-bar and ball penetration in soft-stiff, stiff-soft, soft-stiff-soft, and stiff-soft-stiff clay deposits. Digital images of T-bar and ball penetrating through the layered samples against a transparent window were acquired using a high-speed camera. The PIV technique was adopted to process the image aiming at quantifying the soil
Soil flow mechanisms of full-flow penetrometers in layered clays – PIV analysis in centrifuge test

displacement field resulting from full-flow penetrometer tests, which was then used to investigate the evolution of soil flow mechanisms. Key conclusions are drawn below.

For the T-bar penetration, it was found that

1. Overall a symmetric rotational flow around the half section of the T-bar dominated the behaviour.

2. A trapped cavity mechanism was mobilised above the advancing T-bar in the stiff clay.

3. Close to a soft-stiff interface, a squeezing mechanism was mobilised; and close to stiff-soft interface, significant downwards movement of stiff-soft interface was observed but no trapping of stiff soil plug detected.

4. Compared to the conventional mechanisms used for developing plasticity solutions, the observed evolving mechanisms in thick stiff clay layer and in stratified clays are different, necessitating an improvement of bearing capacity factors accounting for the mobilised failure mechanisms for more accurate interpretation of undrained shear strength from in situ T-bar test data.

For the ball penetration, it was found that

1. Overall a combination of vertical flow, cavity expansion type flow and rotational flow dominated the behaviour.

2. An open cavity with delayed backflow up to the stiff-soft interface in a thin stiff clay layer.

3. Close to a soft-stiff interface, a squeezing mechanism was mobilised; and close to stiff-soft interface, punch-through occurred and hence a stiff soil plug
trapped at the base of the advancing ball penetrated from stiff to soft layer.

4. Different soil flow mechanisms were mobilised in any soil profile compared to the conventional mechanisms used for developing plasticity solutions. It is therefore necessary to improve ball bearing factor based on the mobilised soil flow mechanisms.

2.5 References


Soil flow mechanisms of full-flow penetrometers in layered clays – PIV analysis in centrifuge test


Soil flow mechanisms of full-flow penetrometers in layered clays – PIV analysis in centrifuge test


Table 2-1. Centrifuge test program

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<td>$h_1^*$</td>
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<tr>
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<td>TB1</td>
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</tr>
<tr>
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<td>Ball1</td>
<td></td>
<td></td>
</tr>
<tr>
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<td>TB2</td>
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<tr>
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<td>Ball2</td>
<td></td>
<td></td>
</tr>
<tr>
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<td>TB3</td>
<td>Soft-stiff-soft</td>
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<tr>
<td></td>
<td>Ball4</td>
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</tr>
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</table>

* Model scale
† Prototype scale
Figure 2-1. Push-in penetrometers in field investigation and centrifuge testing
Soil flow mechanisms of full-flow penetrometers in layered clays – PIV analysis in centrifuge test

Figure 2.2. Existing mechanisms for T-bar penetrometer: (a) full-flow mechanism (after Randolph & Houlsby, 1984); (b) pre-embedment less than half diameter (after Murff et al., 1989); (c) pre-embedment larger than half diameter (after Aubeny et al., 2005); (d) shallow failure mechanism and flow-round mechanism (after White et al., 2010); (e) trapped cavity mechanism (after Tho et al., 2011)
Figure 2-3. Existing mechanisms for ball penetrometer: (a) flow-round mechanism (after Randolph et al., 2000); (b) combined mechanism (after Zhou & Randolph, 2011)
Soil flow mechanisms of full-flow penetrometers in layered clays – PIV analysis in centrifuge test

Figure 2-4. Setup of PIV testing in beam centrifuge: (a) photograph before a PIV T-bar test (b) schematic representation (unit: mm)
Figure 2-5. Centrifuge model: (a) model penetrometers; (b) schematic diagram of penetrometers penetration in layered clay
Figure 2-6. Penetration profiles from full penetrometer test: (a) miniature T-bar test in B1~B4; (b) T-bar and ball penetrometer test in bottom stiff layer of B1 and B4
Figure 2-7. Soil flow mechanisms from T-bar penetration in soft-stiff clay (B1, TB1; Table 2-1): (a) $d/D_t = 2.0$; (b) $d/D_t = 5.9$; (c) $d/D_t = 9.8$; (d) $d/D_t = 11.8$; (e) $d/D_t = 13.9$
Soil flow mechanisms of full-flow penetrometers in layered clays – PIV analysis in centrifuge test

Figure 2-8. Schematic diagram of tracked soil elements
Soil flow mechanisms of full-flow penetrometers in layered clays – PIV analysis in centrifuge test

(c)

(d)
Figure 2-9. Displacement paths of soil elements at ~4.4\(D\) above soft-stiff interface during continuous T-bar penetration (B1, TB1; Table 2-1): (a) trajectories of soil elements M1~M5; (b) horizontal and vertical displacements of soil element M1; (c) evolvement of normalised velocity of soil element M1; (d) comparisons of trajectories observed in this study and that from full-flow mechanism (Martin & Randolph, 2006)
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Figure 2-11. Soil flow mechanisms from T-bar penetration in stiff-soft clay (B2, TB2; Table 2-1): (a) $d/D_t = 1.5$; (b) $d/D_t = 4.7$; (c) $d/D_t = 9.7$; (d) $d/D_t = 12.4$; (e) $d/D_t = 14.0$; (f) $d/D_t = 14.7$
Figure 2-12. Limit analysis of T-bar in stiff-soft clay using observed boundary condition shown in Figure 2-11 (with trapped cavity) and comparison against the prediction equation and pre-embedded limit analysis without trapped cavity.
Figure 2-13. Displacement paths of soil elements at ~3.4\(D_t\) above stiff-soft interface during continuous T-bar penetration (B2, TB2; Table 2-1): (a) trajectories of soil elements M1~M5; (b) horizontal and vertical displacements of soil element M1
Figure 2-14. Displacement paths of soil elements at ~0.1D₁ above stiff-soft interface during continuous T-bar penetration (B2, TB2; Table 2-1): (a) trajectories of soil elements N1~N5; (b) horizontal and vertical displacements of soil element N1
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Figure 2-15. Soil flow mechanisms from T-bar penetration in soft-stiff-soft clay (B3, TB3; Table 2-1): (a) $d/D_t = 4.3$; (b) $d/D_t = 9.9$; (c) $d/D_t = 12.7$
Soil flow mechanisms of full-flow penetrometers in layered clays – PIV analysis in centrifuge test

(a) 1st layer (stiff clay)
(b) 1st layer (stiff clay)
(c) 1st layer (stiff clay)
(d) 1st layer (stiff clay)
(e) 1st layer (stiff clay)
(f) 1st layer (stiff clay)
Figure 2-16. Soil flow mechanisms from T-bar penetration in stiff-soft-stiff clay (B4, TB4; Table 2-1): (a) $d/D_t = 3.5$; (b) $d/D_t = 4.6$; (c) $d/D_t = 5.8$; (d) $d/D_t = 6.8$; (e) $d/D_t = 8.3$; (f) $d/D_t = 9.8$; (g) $d/D_t = 11.5$
Figure 2-17. Resistance profile from T-bar penetration in stiff-soft-stiff clay (B4, TB4; Table 2-1)
Figure 2-18. Soil flow mechanisms from ball penetration in soft-stiff clay (B1, Ball1; Table 2-1): (a) $d/D_b = 0.4$; (b) $d/D_b = 3.9$; (c) $d/D_b = 8.4$; (d) $d/D_b = 9.0$; (e) $d/D_b = 10.2$
Figure 2-19. Typical streamline of ball penetration mechanism
Figure 2-20. Displacement paths of soil elements at ~2.4D_b above soft-stiff interface during continuous ball penetration (B1, Ball1; Table 2-1): (a) trajectories of soil elements M1~M5; (b) horizontal and vertical displacements of soil element M1
Figure 2-21. Displacement paths of soil elements at \( \sim 0.1D_b \) above soft-stiff interface during continuous ball penetration (B1, Ball1; Table 2-1): (a) trajectories of soil elements N1–N5; (b) horizontal and vertical displacements of soil element N1
Figure 2-22. Soil flow mechanisms from ball penetration in stiff-soft clay (B2, Ball2; Table 2-1): (a) $d/D_b = 8.2$; (b) $d/D_b = 9.2$; (c) $d/D_b = 10.0$; (d) $d/D_b = 10.7$
Soil flow mechanisms of full-flow penetrometers in layered clays – PIV analysis in centrifuge test

Figure 2-23. Displacement paths of soil elements at ~3.1\(D_b\) above stiff-soft interface during continuous ball penetration (B2, Ball2; Table 2-1): (a) trajectories of soil elements M1~M5; (b) horizontal and vertical displacements of soil element M1
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CHAPTER 3  SOIL FLOW MECHANISMS AROUND CONE PENETROMETER IN LAYERED CLAY SEDIMENTS – PIV IN CENTRIFUGE TEST

ABSTRACT

Cone penetrometer test (CPT) is widely used for in-situ site investigations and for establishing direct penetrometer to foundation (e.g. pile) or anchor (e.g. suction caisson) design correlations. This chapter focuses on the soil flow mechanisms during the continuous penetration of a cone penetrometer in layered clays. A series of centrifuge tests was conducted with the cone penetrometer penetrating through soft-stiff, stiff-soft, soft-stiff-soft, and stiff-soft-stiff clay profiles. Particle image velocimetry (PIV) allowed accurate resolution of the soil flow mechanism around the cone where a half cone model was penetrated into layered clays against a transparent window. The observed soil movement was compared with both previous observations from pile/cone and shallow strain path method (SSPM) prediction. The comparison with SSPM results shows that, the SSPM can provide reasonable evaluations on maximum lateral and vertical displacements even though the upheave movement is overestimated. The effect of soil layering on the failure mechanisms was studied extensively by exploiting soil flow mechanisms and soil displacement paths at various distances from the cone centreline and the soil layer interface. The reported characteristics of cone penetration in layered soils provided in-depth understanding of cone penetration responses that will lead to the development of mechanism-based theoretical model for cone penetration in layered fine-grained soils.
Chapter 3

3.1 Introduction

Cone penetrometer test (CPT) is not only one of the most popular in-situ tools for site investigations, but also regarded as an effective tool for designing pile, suction caisson and torpedo anchors due to the resemblances of the corresponding penetration process (DNV, 1992; Borghi et al., 2001; Andersen et al., 2008; Hossain et al., 2015; Koh et al., 2017). Due to its wide application, the penetration of the cone penetrometer has been studied extensively to establish the relationship between soil strength and measured resistance (Baligh, 1975; Teh & Houlsby, 1991; Van den Berg et al., 1996; Yu & Mitchell, 1998; Yu et al., 2000; Lu et al., 2004; Walker & Yu, 2010; Vermeer, 2013; Gupta et al., 2015; Ma et al., 2017). Since the measured resistance is inherently linked to the mobilised soil failure mechanisms around the cone, various methods have been adopted to predict, measure or visualise the soil displacement.

3.1.1 Cone penetration mechanisms from theoretical solutions

Theoretical prediction of soil displacement from the cone penetration mostly falls into the regime of the cavity expansion theory or the strain path method. The lateral displacement induced by the cone penetration can be simplified as a two-dimensional plain strain problem and evaluated with the cylinder cavity expansion theory (Randolph & Wroth, 1979). A more sophisticated analytical model, the strain path method (SPM), has been used to estimate the deformation area around the cone at deep penetration, which simplifies soil as a viscous fluid flowing around a stationary
penetrometer located within the flow (Baligh, 1985). Sagaseta et al. (1997) improved the SPM accounting for stress free ground surface and named as shallow strain path method (SSPM), which allows the cone penetration to be simulated from the soil surface. However, these methods do not take into account the effect of soil layering.

3.1.2 Cone penetration mechanisms from numerical analyses

Numerical analysis techniques have been used to explore the soil failure mechanisms in layered soils. Van den Berg et al. (1996) presented an Eulerian analysis of the cone penetration tests in multilayer soils. Based on a finite difference approach, analyses were carried out by Ahmadi & Robertson (2005). A linear elastic analysis studying the effects of soil layering on penetration resistance was carried out by Vreugdenhil et al. (1994). For cone penetration in multilayer clays, Walker & Yu (2010) analysed for two layer stiff-soft and three-layer uniform stiff-soft-stiff clays using arbitrary Lagrangian-Eulerian (ALE) method. Large deformation finite element (LDFE) analyses were carried out using the commercial FE package Abaqus/Explicit. Ma et al. (2015, 2017a, 2017b) have explored a much wider range encompassing soft-stiff, stiff-soft, soft-stiff-soft and stiff-soft-stiff clay profiles with LDFE analyses performed using AFENA. The general consensus is that the penetrometer resistance in the vicinity of the interface between the two soil layers depends on the strength and stiffness ratio between the adjacent layers and thickness of the interbedded layer relative to the cone diameter.
3.1.3 Cone penetration mechanisms from physical model tests

Physical direct measurements or visualisations of soil displacements through laboratory testing have been achieved in two ways: (i) penetrating a model full-cone into the body of representative transparent soils (i.e. transparent clay or transparent sand) in a transparent box or a box with a transparent window side, which removed the opacity of natural soils and facilitated the soil motion to be captured by a camera via a laser sheet or markers (Lehane & Gill, 2004; NI et al., 2010; Chini, 2015); (ii) penetrating a model half-cone into natural soil samples against a transparent window of the testing box, allowing for the soil motion to be captured by a camera via the textures added on the soil facing the window (Randolph et al., 1979; Gue, 1984; Mo et al., 2015). The texture was created by coloured flock powders to provide contrasting texture to the fine grain soils for image analysis (Hossain & Randolph, 2010). Then the captured images can be analysed by advanced image analysis techniques of particle image velocimetry (PIV; White et al. (2003)) to quantify the failure mechanisms. The half-object technique with PIV analysis is applied for natural soils. This can eliminate the limitation and dissimilarity of transparent soils compared to natural soils (Liu & Iskander, 2010). In layered sediments, only Mo et al. (2015) carried out a series of centrifuge tests and exposed cone penetration mechanisms in layered sand deposits varying the density of the sand layers by the half-cone model. To the authors’ knowledge, there has no such investigation undertaken of cone penetration in layered clay deposits.
3.1.4 Aim of this chapter

This chapter fills the knowledge gap by presenting physical visual evidence of soil failure mechanisms associated with the cone penetration in layered clay deposits. The soil movements measured in the centrifuge tests and quantified through PIV analyses are compared with those from the theoretical shallow strain path method (SSPM). The effect of soil layering on the soil failure mechanisms is revealed through both the evolving soil flow patterns and soil particle displacement paths at various distances from the layer interface.

3.2 Centrifuge modelling

All centrifuge tests were carried out at 50g in the beam centrifuge at The University of Western Australia (Randolph et al., 1991). The experimental set-up is shown in Figure 3-1. A purposely-designed PIV strongbox with a Plexiglas window was built to allow the observation of soil deformations through the window. The PIV strongbox has an internal size of 337 mm (length) × 100 mm (width) × 299 mm (depth), which was fitted tightly at one side of a beam standard strongbox with a camera mounted on the other side for image capturing. A 20 mm water layer was maintained above the soil sample during testing to ensure saturation.

3.2.1 Model penetrometer

A model half-cone penetrometer was fabricated by dissecting a 10 mm diameter cylindrical duraluminum bar \( D_c = 0.5 \) m in prototype) and machining the apex tip
angle to 60° (Figure 3-2a). A rubber O-ring was fitted along the periphery of the flat face of the model. This was to achieve a good contact of the cone-face with the window while penetrating the half-cone against the window avoiding any soil ingress between the cone face and the window. The surfaces of the model were relatively smooth as they were anodised and not sand blasted.

Figure 3-2b shows the schematic display of the cone penetrometer penetration in layered clay, where $d$ is the penetration depth of the cone tip from the soil surface, $t$ is the distance of the cone tip to the underlying layer interface. $s_{ui}$, $h_i$ and $\gamma'_i$ represent the undrained shear strength, height, and soil density of the corresponding layer, $i = 1, 2, 3$. The layer shaded with light grey is to represent the stiff clay layer, and this is consistent with all the mechanism figures in this chapter.

The model cone was penetrated into the soil at a constant rate of $v = 0.3$ mm/s, ensuring undrained conditions (Finnie & Randolph, 1994) with dimensionless velocity $vD_c/c_v = 37 > 30$ ($c_v = \sim 2.6$ m$^2$/year), and allowing adequate image capturing frequency for PIV analysis at the same time.

All half-cone PIV tests were performed in the middle section of the window (Figure 3-1b) within the camera field of view. In order to obtain the cone resistance profile in layer sediments, a standard full-cone penetrometer of the same diameter as the PIV model (i.e. $D_c = 10$ mm) is adopted and tested at intervals during each test (Figure 3-1b). The final penetration depth was around 220 mm, leaving a 50 mm (i.e. $5D_c$)
clearance from the bottom boundary (soil sample height = 270 mm) and a 90 mm ($9D_c$) clearance from the back wall boundary. These clearance distances were to minimise any boundary influence, which was confirmed by the recorded penetration resistance profiles, (with no significant increase when approaching its final penetration depth) and observed soil failure mechanisms. Further, from an extensive investigation, Ullah et al. (2014) reported that, in clay, a distance of 0.7~1.0 diameter of the penetrating object is sufficient for avoiding the influence of the boundary.

3.2.2 Sample preparation and soil strength characterisation

The cone penetration tests were performed on layered specimens of kaolin clay, with the engineering properties given by Stewart (1992). A homogeneous de-aired slurry was prepared by mixing commercially available kaolin clay powder with water at 120% water content in a ribbon-blade mixer with vacuum pump. Clay samples with uniform strength profile were prepared by performing consolidation at 1g on the laboratory floor in two separate 625 mm long × 390 mm wide × 325 mm deep cuboid strongboxes. One-dimensional consolidation was performed in stages, up to final pressures of 400 kPa and 100 kPa to produce comparatively stiff and soft samples. Test specimens were prepared by cutting the pre-consolidated samples according to the size of the PIV strongbox and planned layers thickness, and then sliding the layers into the PIV strongbox according to planned stratification (as in Table 3-1). To add texture to the white kaolin clay for PIV analysis and to track the deformation of different layers, green and black flock powders were sprinkled on the stiff and soft clay layers
respectively on the sides facing the window (Dingle et al., 2008; Hossain & Randolph, 2010; Hossain, 2014).

Four boxes of test specimens were prepared varying stratification as soft-stiff, stiff-soft, soft-stiff-soft, stiff-soft-stiff (B1~4; Table 3-1). In addition, a box was prepared with single layer soft clay (B0; Table 3-1). The details of layered soil strength determination can be found in the previous chapter (2.3.1) of the thesis, with the soft layer is ~7.5 kPa and stiff layer ~29.1 kPa. The effective unit weights ($\gamma'$) of the soft and stiff layers were ~7.0 kN/m$^3$ and ~7.4 kN/m$^3$ respectively.

### 3.2.3 Image capture and PIV analysis

The experimental set up is shown in Figure 3-1. In order to reduce friction, white petroleum jelly was used to smoothen the flat face of the half cone and rubber O-ring. A two-dimensional actuator was mounted on the sides of the standard beam strongbox, and above the PIV strongbox. The half cone was then fitted with the actuator and pushed in-flight into the soil sample against the window. Pictures were taken continuously by the camera mounted at right angle to the window. Two LED lights placed at the top and bottom just in front of the PIV box were used to provide sufficient light for the pictures.

A Prosilica GC2450C digital still camera operating in continuous shooting mode was used to capture the images of the penetrometer and the surrounding soil throughout the tests. The 5 megapixel digital images were taken at 0.05 mm/picture (i.e. 6 Hz) for
Soil flow mechanisms around cone penetrometer in layered clay sediments – PIV in centrifuge test

each test. A 50-mm grid consisting of 24 control points (black dot on a white background) was installed within the Plexiglas window. A centroiding technique, based on the coordinates of these control points from reference axes, was implemented for photogrammetric corrections and transferring image space to object space. In this way, any in-flight image-space displacement of the control markers incurred by the movement of the camera during testing was accounted for. Full details of the PIV and photogrammetric analysis are described in White et al. (2003) and Stanier et al. (2015).

3.3 Results and discussion

This section discusses the effect of soil layering on the soil failure mechanisms around the cone penetrometer. A range of stratification commonly encountered in offshore fine grained seabed sediments (Osborne et al., 2011) were considered as soft-stiff, stiff-soft, soft-stiff-soft and stiff-soft-stiff clay. The results from testing on single layer soft clay (B0) are presented first for comparing with existing mechanisms.

3.3.1 Cone penetration in single layer clay and comparison with SSPM theory

Figure 3-3a illustrates the soil flow mechanism at $d/D_c = 13.2$ for the cone penetration in single layer clay (Box B0, cone test CPT0; Table 3-1). The incremental displacement field (vectors on the left side of Figure 3-3a) was quantified by successive images with a cone penetration increment of $0.22D_c$. The corresponding contours are displayed on the right side of Figure 3-3a. The vectors are scaled up 6 times for a clear view and the values of contour are normalised by the cone penetration
incremental depth. The contour levels in the figure includes 0.5, 0.45, 0.4, 0.35, 0.3, 0.25, 0.2, 0.15, 0.1 and 0.05 (i.e. the label of 0.5~0.05 in Figure 3-3a). This style is kept for all following graphs of soil flow mechanisms in the rest of this chapter.

It can be observed that the soil displacement zone is a bulb mainly around the cone tip and expands radially somewhat perpendicular to the face of the cone tip. This feature is consistent with the (i) observed mechanism from a cone penetration in both loose and dense sands (Mo et al., 2015) (Figure 3-3b), (ii) mechanism from a cone penetration LDFE analysis in clay (Ma et al., 2017) (Figure 3-3c), (iii) SSPM theory prediction (Figure 3-3d). For the cone, the closest contour line to the cone is 0.5, showing that the soil near the conical face moves at half of the cone penetration rate. However, for the cylindrical base of the T-bar or spherical base of the ball penetrometer, the soil right beneath the penetrometer moves with the penetrometer at roughly its full penetration rate (i.e. within 0.9 contour line; Figure 2-7). The outer contour line represents a soil displacement at 5% of the cone penetration depth, which defines the soil displacement zone. The size of this zone can be measured in Figure 3-3a as $1.1D_c$ downward (from the cone tip) and $1.2D_c$ outward (from the cone centre line). The upper boundary of the influence zone is in accordance with a line with an inclination of ~40° to the vertical (Figure 3-3a). This is close to the 45° obtained from the test by Mo et al. (2015) in loose sand (Figure 3-3b).

The contours of soil displacement ratio to the cone incremental penetration depth predicted by SSPM (Sagaseta et al., 1997) with the same penetration depth (prototype)
are depicted in Figure 3-3d. It is noted that the SSPM is built on the simple pile solution that originally proposed by Baligh (1975) and, therefore, a round tip was considered. In general, it shows a reasonable agreement between the present PIV analysis with a conical tip and SSPM analysis with a round tip for the soil displacement zone below the cone tip. However, by SSPM, the upper boundary of the soil displacement zone is over predicted above the cone shoulder. This may be attributed to the fact that SSPM simplifies the soil as an ideal flow and excludes soil overburden pressure, cone surface friction etc., hence overestimates the soil upward movement. For the cone with a round tip (SSPM, Figure 3-3d), the soil displacement vectors point in the radial direction from the centre of the round tip; while for the cone with a conical tip, the soil displacements somewhat perpendicular to the face of the cone tip (PIV, Figure 3-3a).

3.3.2 Cone penetration in layered sediments

3.3.2.1 Cone penetration resistance profiles

Figure 3-4a–b display the resistance profiles from full-cone penetration tests in two layer (Boxes B1 and B2 in Table 3-1) and three layer (Boxes B3 and B4 in Table 3-1) deposits, respectively. The net cone resistance is calculated by deducting the overburden pressure and divided by projection area (details referring to Chapter 4), and the cone penetration depth is normalised as \( d/D_c \). For stiff-soft layering (B2), the cone resistance decreases abruptly when the cone tip touches the soil layer interface followed by a relatively gradual decrease in the soft layer. In contrast, for soft-stiff
layering (B1), there is a sudden increase in resistance when the cone tip touches the soil layer interface followed by a consistent increase rate in the stiff soil layer till the resistance reaches its limit. The transition depth from the top layer to the bottom layer is relatively faster for soft-stiff soil layers (B1) than that for stiff-soft soil layers (B2).

In the soft 1st layer, the thickness of \( h_1/D_c = 18.5 \) (B1 in Figure 3-4a) is sufficient to mobilise the full (ultimate) penetration resistance of that layer. In the stiff 1st layer, the thickness of \( h_1/D_c = 18.6 \) (B1) seems to be not sufficient to mobilise the full (ultimate) resistance of that layer. The full resistance may be significantly higher as can be seen from the resistance in the stiff 2nd layer (B1) and stiff 3rd layer (B4). The four resistance profiles show that the net resistance of the soft layers fall into the range of 80~120 kPa, and the stiff layer fall into 225~320 kPa. Some softening in the top layer may account for the discrepancy. The interbedded stiff layer with \( h_2/D_c = 8.0 \) (B3) is apparently not sufficient to reach the ultimate resistance of that layer, with the ultimate resistance lower than any of the stiff layers.

The soil movements during the process of cone passing through the layer interfaces will be illustrated in detail in the following section with half-cone PIV analysis results at different depths, which are also marked on Figure 3-4 with filled triangle or circle at corresponding resistance profile.

**3.3.2.2 Cone penetration in soft-stiff clay: soil flow mechanism**

Figure 3-5 illustrates the soil flow mechanisms of the cone penetration in soft-stiff clay \( (h_1/D_c = 18.5; B1, CPT1; Table 3-1) \). It can be seen that during initial penetration
in the top soft layer \((d/D_c = 11.8; \text{Figure 3-5a})\), the flow mechanism is similar to that in uniform soft sample (Figure 3-3a). With further penetration as the cone approaches the interface \((d/D_c = 18.1; \text{Figure 3-5b})\), the soil flow is mostly restricted in the 1\(^{st}\) soft layer with minimal flow in the 2\(^{nd}\) stiff layer. The height of the displaced zone is therefore reduced. With the tip just entering the stiff clay \((d/D_c = 18.5; \text{Figure 3-5c})\), the soil flow in the 1\(^{st}\) layer is displaced laterally, leading to mobilisation of a squeezing mechanism. This is very similar to the squeezing mechanism mobilised with a T-bar approaching a stiff clay layer (Figure 2-7). Because of this lateral squeezing, when the cone tip penetrates fully into the 2\(^{nd}\) layer \((d/D_c = 19.6; \text{Figure 3-5d})\), the layer interface deforms only moderately and no soft clay is dragged down under the interface. The soil flow becomes fully confined within the 2\(^{nd}\) (stiff) layer. This has led to a sharp increase in penetration resistance in the 2\(^{nd}\) layer immediately after the soft-stiff layer interface (see Figure 3-4a).

A series of soil elements are tracked to study the lateral and vertical movements induced by the cone penetration. The original location of the soil elements, as well as the cone penetration depth at the start \((y = y_1)\) and end \((y = y_2)\) of tracking, are defined in Figure 3-6. To understand the effect of layering on soil displacements, the soil elements at various distances from the layer interface \((t)\) are studied. At each distance \(t\), a series of soil elements with various distances from the penetration centreline \((x)\) are tracked.
For soil elements far away from the soil layer interface, a series of soil element with \( x \) varying from \( 0.75D_c \) to \( 6D_c \) are tracked as the cone penetrates from \( 3D_c \) above the soil element level (i.e. \( y_1 = -3D_c \)) to \( 2D_c \) under it (i.e. \( y_2 = 2D_c \)). This is regarded as a case with little influence of the 2\textsuperscript{nd} layer due to the large distance away from the interface of \( t = 4.5D_c \). The tracked trajectories are shown in Figure 3-7a. Generally, they all show a downward-outward movement as the cone passes the soil element. As expected, the soil element closer to the cone travels deeper and wider distances. At the end of the displacement path, a slight upward movement is recorded for \( x \leq 1D_c \). This is consistent with the observation of a flat-end cone in transparent soil by Lehane & Gill (2004), where upward movement was observed for \( x \leq 0.92D_c \). However, their observed upward movement at the end of the path in transparent soil is greater than that obtained from this study in clay. The greater upward soil movement observed in transparent soil may be due to the low overburden pressure from the low unit weight of the transparent soil, i.e. amorphous silica (Ganiyu et al., 2016), and shallower soil element embedment depth (114 mm in Lehane & Gill (2004) vs. 7 m in this study).

As the cone approaches (\( y < 0 \)) and leaves (\( y > 0 \)) the soil element at \( x = 0.75D_c \), the evolution of normalised incremental lateral (\( \Delta x/D_c \)) and vertical (\( \Delta y/D_c \)) displacements are plotted in Figure 3-7b as a function of the cone elevation (i.e. \( y/D_c \), negative \( y \) means the position of cone above the original soil element elevation). When the cone elevation is between -\( 2.5D_c \) and \( 0.5D_c \), the soil downward displacement accumulate earlier and faster than the outward movement. Both \( \Delta x/D_c \) and \( \Delta y/D_c \) increase sharply
when the cone passes the soil element with $y/D_c = -0.5~1.0$, which is termed as ‘active zone’. When the cone is at $1.0D_c$ below the original level of the soil element, both lateral and vertical displacement attains to a plateau i.e. the soil movement diminishes. The soil element rests at $0.11D_c$ outwards and $0.08D_c$ downwards from its original location. The slopes of the curves are the displacement ratio of the soil element to the cone penetration velocity. Since the cone has a constant penetration velocity, the slopes of the curves represent the soil element velocity. It is apparent that the peak velocities of the soil element in both $x$ and $y$ directions occur at about $y = 0$, i.e. when the cone tip reaches the original soil element level.

The corresponding theoretical soil displacement trajectories from SSPM are shown in Figure 3-8 for comparison. The displacement paths for all elements form an arc showing downward-outward followed by upward-outward trend. By comparing with the measured displacement in Figure 3-7a, theoretical displacements are shallower and much more profound in upward movement. Possible explanation might be: (i) rigid perfectly-plastic material (inviscid fluid) is considered in SSPM, and as such no downward elastic deformation can be reflected, and (ii) no overburden pressure is considered, allowing more upward movement.

The measured ultimate lateral and vertical displacements for soil elements at various distances from the cone penetration track are illustrated in Figure 3-9a and Figure 3-9b together with the experimental results on cone or pile penetration from the literature.
(Cooke & Price, 1973; Randolph et al., 1979; Francescon, 1983; Gue, 1984; Lehane & Gill, 2004). The cone or pile model geometry and soil sample information of these experimental programmes are included in the legend, and more details can be found in Gill & Lehane (2001) and Lehane & Gill (2004). It is evident that the tracked cone penetration depth in Figure 3-7a covers the full displacement process of the soil elements selected. Thus the ultimate displacements of the soil elements in the current study agree very well with the existing data, where the soil displacement tracking was made for the full penetration of the object (i.e. cone or pile), despite the variation in experimental procedure, penetrating object and soil material used.

The predictions by two theoretical methods—cavity expansion theory (Randolph et al., 1979) and SSPM (Sagaseta et al., 1997)—are also included in Figure 3-9 for comparison. The cylinder cavity expansion theory can only predict lateral outward movement, which is slightly larger than that measured from this study. SSPM provides good estimation of the lateral displacement and maximum downward movement throughout the whole measured range, but underestimates the ultimate vertical displacement especially for $x/D_c < 1$ due to the outward-upward movement predicted after the outwards-downwards movement (see Figure 3-8).

For soil elements near the soil layer interface, the tracked trajectories of soil elements at $t = 0.1D_c$ with various distances from the penetration track are shown in Figure 3-10. The effect of squeezing from the underlaying stiff layer, displayed in Figure 3-5b–c,
can be quantified by comparing Figure 3-10 with Figure 3-7. It can be seen that, the presence of the underlying stiff layer (Figure 3-10) has forced the soil elements more outward and less downward. For the element at \( x/D_c = 0.75 \), the ultimate \( \Delta x/D_c = 0.13 \), \( \Delta y/D_c = 0.07 \) in Figure 3-10; and ultimate \( \Delta x/D_c = 0.11 \), \( \Delta y/D_c = 0.08 \) in Figure 3-7, which means that the maximum lateral displacement is increased by 18% and the maximum downward movement is reduced by 14%.

3.3.2.3 **Cone penetration in stiff-soft clay: soil flow mechanism**

The evolvement of soil flow mechanisms with the advancement of the cone in stiff-soft clay is shown in Figure 3-11 (\( h_1/D_c = 18.6 \); B2, CPT2; Table 3-1). The soil flow mechanism in stiff clay (\( d/D_c = 12.4 \); Figure 3-11a) is very similar to that in soft clay (Figure 3-3a and Figure 3-5a), with the outer contour reaches 1\( D_c \) (compared to 1.1\( D_c \) in soft clay) below the cone tip and 1.2\( D_c \) (compared to 1.2\( D_c \) in soft clay) from the penetration centreline. The upper boundary of the influence zone inclines ~45° to the vertical, which is slightly higher compared to 40° in soft clay (see Figure 3-3a), but matches with 45° inclination obtained by Mo et al. (2015) from testing in loose sand.

When the cone approaches the soil layer interface (\( d/D_c = 17.8 \); Figure 3-11b), the soil flow directs mostly downward and laterally outward in the 2\(^{nd} \) layer since it is easier to move the soft soil underneath. This resembles the ‘punch-through’ mechanism observed by Hossain and Randolph (2010) for spudcan penetration in stiff-soft clay deposits. As mentioned before, here the term ‘punch-through’ is only for depicting the enhance soil downwards movement when approaching the bottom soft layer, because
displacement-control penetrometer is not at the risk of punching-through. As the cone tip penetrates into the soft layer \((d/D_c = 18.8; \text{Figure 3-11c})\), a significant deformation of the layer boundary occurs by the large downward soil movement. Some stiff soil material is being dragged down with the cone. After the cone tip passes through the layer boundary \((d/D_c = 20.7; \text{Figure 3-11d})\), the soil flow is fully confined in the soft layer. After the cone tip penetration into the 2\(^{nd}\) layer, a certain penetration depth is required for the soil mobilisation zone to be detached from the 1\(^{st}\) layer. The mobilised mechanisms in Figure 3-11c–d, have led to a more gradual decrease in penetration resistance profile in the 2\(^{nd}\) (soft layer) as can be seen in Figure 3-4. Due to the pointy tip of the cone, no stiff soil plug is trapped under the cone penetrometer, unlike the spherical ball penetrometer illustrated in Chapter 2.

Similar to the soft-stiff clay study, two groups of elements located originally at \(~4.5D_c\) (away from the layer interface) and \(~0.1D_c\) (near the layer interface) above the layer interface were tracked as the cone penetrates from \(3D_c\) above the soil element level (i.e. \(y_1 = -3D_c\)) to \(2D_c\) under it (i.e. \(y_2 = 2D_c\)). Their trajectories are shown in Figure 3-12. The relative trend between Figure 3-12b and Figure 3-12a (with and without the underlying layer influence) is reversed of that between Figure 3-10 and Figure 3-7. Compared to Figure 3-12a, the downward movement of Figure 3-12b is augmented by \(~19\%) and the outward movement is shortened by \(~24\%). This is because the underlying soft layer of stiff-soft clay deposit attracts the soil downward movement as opposed to restriction by the underlying stiff layer of soft-stiff clay deposit.
The displacements of the soil elements away from the interface (Figure 3-12a) can be compared to those in soft layer (Figure 3-7a) (both with ~4.5\(D_c\) from the underlying layer). For the soil element at \(x = 0.75D_c\), it can be noted that the ultimate vertical downward displacement (\(\Delta y\)) in stiff clay is significantly higher, for example \(\Delta y = 0.14D_c\) in stiff layer versus \(0.08D_c\) in soft clay. A direct comparison is made in Figure 3-13 showing the development of soil vertical displacement with the cone penetration in soft-stiff (CPT1) and stiff-soft (CPT2) clay. For both soil elements, the vertical displacement initiates from cone at ~3\(D_c\) above the original level and attains to the peak or ultimate displacements at around ~1\(D_c\) below the original level although the rate of development and the ultimate displacement are remarkably higher in stiff clay (CPT2). Even though the displacement zones are only slightly different (40° for CPT1 and 45° for CPT2), as described in the previous section, the difference has accumulated with continuous penetration, resulting in an obvious longer trajectory. This may be an evidence that soil parameters (including soil layer profiles) have an accumulated influence on the mobilised flow mechanism zone, especially on the soil displacements. This is contrary to the assumption adopted in SPM/SSPM analysis where soil displacement path is insensitive to soil parameters. This influence should be taken into account for assessing various geotechnical problems such as spudcan-pile interaction, pile-pile interaction and so on.
3.3.2.4 Cone penetration in three layer clay: soil flow mechanism

Figure 3-14 presents the mechanisms of the cone penetration in soft-stiff-soft clay at 4 critical depths (B3, CPT3; Table 3-1). For cone in the 1st-2nd (soft-stiff) layers, the mechanism is similar to those from CPT1. However, due to the thinner 1st layer for CPT3 ($h_1/D_c = 6.8$ for CPT3 vs $h_1/D_c = 18.5$ for CPT1) (see Table 3-1), when the cone passes through the interface of 1st-2nd (soft-stiff) layers, the overburden pressure (i.e. $\gamma_1'h_1$) for CPT3 is much lower than that for CPT1. The lower overburden pressure for CPT3 near the interface of 1st-2nd layers has led to more lateral movement of the soft soil in the 1st layer. As such, by comparing Figure 3-14a with Figure 3-5b, the soil displacement zone (i.e. 0.05 contour line) is wider (outward) and higher (upward) for CPT3 in Figure 3-14a.

Once the cone has passed the interface of 2nd-3rd (stiff-soft) layers, some similarity can be observed between the mechanism from CPT3 (Figure 3-14c and Figure 3-14d) and those from CPT2 (Figure 3-11b and Figure 3-11d) as the strength ratio and overburden pressure at the interface level are similar ($h_1/D_c = 14.8D_c$ for CPT3 vs $h_1/D_c = 18.6D_c$ for CPT2, see Table 3-1). However, by inspecting Figure 3-11d and Figure 3-14d, it can be seen that, in Figure 3-11d for CPT2, the stiff soil is dragged downwards into the bottom soft layer locally (i.e. adjacent to the cone); in Figure 3-14d for CPT3, the stiff soil bends downward more evenly. This is possibly because the thinner stiff layer for CPT3 behaves more like a beam. Due to the more even bending of the thin stiff
layer in Figure 3-14d, the soil displacement zone is deeper and more spread in the bottom soft layer than that in Figure 3-11d.

Figure 3-15 displays the soil flow mechanisms for the cone penetration in stiff-soft-stiff clay (B4, CPT4; Table 3-1). Similarly, when the cone passes the interface of stiff-soft (i.e. 1st-2nd) layers, the stiff soil is dragged downwards into the 2nd soft layer (Figure 3-15b–c). And the squeezing mechanism in the soft soil is observed when the cone approaches the soft-stiff (2nd-3rd) layer interface (Figure 3-15d).

In summary, as the cone passes the soft-stiff layer boundary, a squeezing mechanism in the soft layer dominates the behaviour with minimal deformation of the layer boundary. This squeezing mechanism is more profound when the overburden pressure is relatively low (i.e. the layer interface is relatively shallow). However, as the cone passes the stiff-soft layer boundary, a ‘punch-through’ mechanism dominates the behaviour, leading to significant downward deformation of the layer interface. Due to the pointy tip of the cone, regardless of the layers strength ratio, no soil plug from the upper layer can be trapped down into the lower layer after the shoulder of the cone passes the interface.

3.4 Concluding remarks

This chapter has reported experimental results from a series of centrifuge model tests investigating the evolution of soil failure mechanisms during a cone penetrometer penetration in single, two layer and three layer clay deposits. The extensive study has
Chapter 3

encompassed soft-stiff, stiff-soft, soft-stiff-soft and stiff-soft-stiff profiles as they are commonly encountered in the hydrocarbon active seabed sediments. The following conclusions can be drawn from the presented results.

For a cone penetrating into a single uniform clay layer, the soil deformation concentrates mainly below the cone shoulder, with a minimal extension above the shoulder. The soil displacement measured in this study falls in the range from previous studies. The comparison with SSPM results shows that, the SSPM can provide reasonable evaluations on maximum lateral and vertical displacements even though the upheave movement is overestimated, underestimating the ultimate displacement.

For a cone penetrating into layered clays, the layering effects can be summarised as:

(1) When the cone passes a soft-stiff layer interface, the laterally outward soil flow in the soft layer just above the interface is apparent, which is termed as squeezing mechanism. More profound squeezing mechanism is associated with relatively low overburden stress at the interface level, leading to minimal deformation of the interface;

(2) When the cone passes a stiff-soft layer interface, the layer interface moves downward, with the stiff soil adjacent to the cone is dragged into the soft layer;

(3) For all investigated layered deposits, no soil plug was trapped at the base of the advancing cone due to the pointy shape of the cone base.
3.5 References


Chapter 3


Soil flow mechanisms around cone penetrometer in layered clay sediments – PIV in centrifuge test


Table 3-1. Summary of centrifuge tests

<table>
<thead>
<tr>
<th>Box</th>
<th>Test</th>
<th>Soil description</th>
<th>Layer thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$h_1^*$&lt;sub&gt;mm&lt;/sub&gt;</td>
</tr>
<tr>
<td>B0</td>
<td>CPT0</td>
<td>Soft clay</td>
<td>250</td>
</tr>
<tr>
<td>B1</td>
<td>CPT1</td>
<td>Soft-stiff</td>
<td>185.4</td>
</tr>
<tr>
<td>B2</td>
<td>CPT2</td>
<td>Stiff-soft</td>
<td>185.9</td>
</tr>
<tr>
<td>B3</td>
<td>CPT3</td>
<td>Soft-stiff-soft</td>
<td>68.4</td>
</tr>
<tr>
<td>B4</td>
<td>CPT4</td>
<td>Stiff-soft-stiff</td>
<td>77.4</td>
</tr>
</tbody>
</table>

*Model scale*
Figure 3-1. Centrifuge test setup: (a) photo; (b) schematic diagram with dimensions (unit: mm)
Figure 3-2. Half cone model for PIV testing: (a) photos; (b) schematic diagram of cone in layered sediment
Soil flow mechanisms around cone penetrometer in layered clay sediments – PIV in centrifuge test

(a)

(b)
Figure 3-3. Soil flow mechanism of cone penetration in soft clay at $d/D_c = 13.0$
(CPT0, Table 3-1): (a) PIV analysis in this study; (b) observed mechanism from
a cone penetration in both loose and dense sands (Mo et al., 2015), (c) mechanism
from a cone penetration LDFE analysis in clay (Ma et al., 2017); (d) SSPM theory
prediction
Soil flow mechanisms around cone penetrometer in layered clay sediments – PIV in centrifuge test

Figure 3-4. Resistance profiles from cone penetrometer: (a) two layer clay; (b) three layer clay
Figure 3-5. Soil flow mechanisms of cone penetration in soft-stiff clay (CPT1, Table 3-1): (a) $d/D_c = 11.8$; (b) $d/D_c = 18.1$; (c) $d/D_c = 18.5$; (d) $d/D_c = 19.6$
Soil flow mechanisms around cone penetrometer in layered clay sediments – PIV in centrifuge test

Figure 3-6. Schematic diagram of tracked point for displacement path
Figure 3-7. Displacement paths of soil elements at ~4.5$D_c$ above soft-stiff interface during continuous cone penetration from $y_1 = -3D_c$ to $y_2 = 2D_c$ (CPT1; Table 3-1): (a) trajectories of soil particle with varying distance from penetration track; (b) development of horizontal and vertical displacements of soil element $x = 0.75 D_c$ with cone penetration
Figure 3-8. Soil element displacement path predicted by SSPM theory
Figure 3-9. Comparison of soil displacement: (a) lateral; (b) vertical
Figure 3-10. Displacement paths of soil elements at ~0.1\(D_c\) above soft-stiff interface during continuous cone penetration (CPT1; Table 3-1): (a) trajectories of soil particle with varying distance from penetration track; (b) development of horizontal and vertical displacements of soil element \(x = 0.75D_c\) with cone penetration
Figure 3-11. Soil flow mechanisms of cone penetration in stiff-soft clay (CPT2, Table 3-1): (a) $d/D_c = 12.4$; (b) $d/D_c = 17.8$; (c) $d/D_c = 18.8$; (d) $d/D_c = 20.7$
Figure 3-12. Soil element displacement path with varying distance from stiff-soft interface (CPT2, Table 3-1) (a) $t/D_c = 4.5$; (b) $t/D_c = 0.1$
Figure 3-13. Comparison of soil element vertical displacement development in CPT1 and CPT2 for soil element originally positioned at $t = 4.5D_c$ and $x = 0.75D_c$.
Figure 3-14. Soil flow mechanisms of cone penetration in soft-stiff-soft clay (CPT3, Table 3-1): (a) $d/D_c = 6.5$; (b) $d/D_c = 8.3$; (c) $d/D_c = 14.3$; (d) $d/D_c = 17.1$
Figure 3-15. Soil flow mechanism of cone penetration in stiff-soft-stiff clay (CPT4, Table 3-1): (a) $d/D_c = 6.9$; (b) $d/D_c = 8.0$; (c) $d/D_c = 9.0$; (d) $d/D_c = 14.8$; (e) $d/D_c = 16.3$
CHAPTER 4  AMBIENT PRESSURE CALIBRATION
FOR CONE PENETROMETER TEST: NECESSARY?

ABSTRACT

The cone tip load cell needs to be calibrated before the cone penetration test (CPT) data in soil can be used to interpret the soil properties. The cone tip load cell is routinely calibrated through cone tip axial loading test. In this study, the CPTs were conducted in fully saturated layered clay deposits in a centrifuge. The net cone resistance was found to be negative by using the calibration factor obtained through axial loading test. This is illogical. Further CPT calibration was performed in water in the centrifuge under 150 times earth gravity. The CPT calibration factor under ambient pressure in water was found to be different from that obtained through axial loading test. By using both calibration factors from axial load test and under ambient pressure, the net cone resistance was recalculated. The interpreted soil undrained shear strength using the recalculated net cone resistance was consistent with that from a parallel ball penetrometer test. Since the soil overburden pressure acts on the cone tip as an ambient pressure, the cone tip load cell should be calibrated under ambient pressure to provide logical interpretation of CPT data.
4.1 Introduction

Due to the high cost of obtaining high quality soil samples offshore, field test becomes a popular choice to determine the geotechnical properties of seabed sediments. Cone penetrometer test (CPT) is the commonly adopted in-situ test, which is mainly due to its superior feature of providing a continuous resistance profile, compared to vane shear test. As shown in Figure 4-1, the cone penetrometer comprises a cone-shaped tip and a cylindrical shaft. The standard cone penetrometer used in the field has a base area of 10 cm$^2$ (diameter $D_c = 35.7$ mm), although the diameter can go up to 71.4 mm (40 cm$^2$) (Robertson & Cabal, 2015). There are also some miniature cones ($D_c = 7$ mm; 10 mm) that have been reported for laboratory test applications (Lee, 2009).

The wide use of cone penetration test for soil characterisation in geotechnical practice rises the demand for accurate measurement, which relies on the accuracy of its sensors. The cone penetrometer generally consists of cone tip load cell and sleeve friction load cell. This study focuses on the former. The cone tip load cell is embedded in the shaft just above the shoulder of the cone (as marked in Figure 4-1). The accuracy of the cone tip load cell and its calibration factor are crucial in terms of interpreting the corresponding soil strength. For the normal practice with a cone, zero-load error is a reliable indicator of output stability (ASTM, 2012). Zero-load error refers to zero-load readings (or baseline readings; Mayne, 2007) difference before and after a sounding. Recalibration is only necessary when the zero-load error drifts out of requirement (ASTM, 2012, Robertson & Cabal, 2015), and ASTM (2012) regulates this critical error as 2% full scale output.

To ensure sensible calculation using the measured data, in-house check-up for calibration factor of a load cell is routinely conducted. The recommended method for
Ambient pressure calibration for cone penetrometer test: necessary? In-house calibration check of the cone tip load cell is through axial load test (Lunne et al., 1997), in which the cone penetrometer is loaded/unloaded against a reference load cell (Peuchen & Terwindt, 2014). The detail of the test can be found in Chen & Mayne (1994). Through loading/unloading, the voltage response of the tested cone tip load cell is recorded and the load on the cone is known from the reference load cell. The calibration factor can be obtained through best fitting the data, with the gradient expressed as,

\[ C = \frac{q}{v_t} \]  

(4-1)

where \( v_t \) is the voltage reading of the load cell and \( q \) is the corresponding pressure applied on the load cell, and the pressure-to-voltage ratio is called conversion or calibration factor (\( C \)). A qualified cone should prove good linear behaviour under loading with nonlinearity satisfy certain requirement (ASTM, 2012).

The calibration factor obtained through the axial load test can only reflect the load cell behaviour under a tip resistance. However, when the cone penetrates into soil, it subjects to both tip resistance and ambient pressure (i.e. overburden pressure) (Figure 4-2). In this chapter, firstly an in-house check-up for calibration factor is conducted through axial load test at 1g on the laboratory floor. Then the load gauge behaviour under ambient pressure is examined through a series of hydraulic pressure tests in a centrifuge. The calibration factors are compared, and both factors are used in the CPT data interpretation framework. Finally, the necessity of performing a calibration test under ambient pressure is demonstrated.
4.2 Experiment details

A 10 mm diameter miniature cone penetrometer with a 60° cone tip was considered in this chapter. A tip load cell with 10 MPa capacity was equipped just above the cone shoulder (Figure 4-1b). No pore pressure transducers were equipped on this penetrometer so area ratio correction is not needed. In this section, firstly two types of calibration test are described—the axial load test that considers net tip resistance and hydraulic test that simulates ambient pressure. Then the CPT and ball tests in a layered clay sample are introduced.

4.2.1 Axial load test at 1g

In the axial load calibration test, the cone penetrometer concentrically placed on a high-resolution reference load cell was mounted on a compression machine, as shown in Figure 4-3. Compression loading and unloading was carried out in steps. In each step, sufficient time was allowed to stabilise the response readings, and the stabilised voltage (or bit from the cone tip load cell) and pressure (from the reference load cell) were recorded. Compression pressure was gradually increased up to 2865 kPa (force 224.9 N/total projection area of the cone 78.5 mm²) prior to unloading.

4.2.2 Hydraulic pressure test at 150g

To test cone tip load cell behaviour under ambient pressure, the cone penetrometer was penetrated from the water surface to the designed depth (220 mm in model scale). To achieve a large water pressure, the hydraulic test was conducted under an enhanced gravity environment of 150 times the earth gravity (150g) in a beam centrifuge at The University of Western Australia (Randolph et al., 1991). The experimental set-up is shown in Figure 4-4. The centrifuge strongbox with internal dimensions of 650 mm (length) × 390 mm (width) × 325 mm (depth) was filled with water. The cone
penetrometer was penetrated in water at a constant velocity of 1.15 mm/s, which is the same as the penetration velocity of the cone in the layered clay sample (described later). High-speed data acquisition system (10 Hz) was equipped with the centrifuge to allow in-flight data monitoring and collection. At the final depth of ~220 mm (~33 m in equivalent prototype scale), the maximum prototype hydraulic ambient pressure was 323 kPa. The load cell response of the penetrometer was tested several times to ensure the data consistency.

4.2.3 Centrifuge penetrometer test on layered clays—cone and ball penetrometers

The same cone penetrometer just discussed was used to characterise a soft-over-stiff layered clay deposit at an acceleration level of 50g. The soft and stiff clay layers were produced from kaolin clay slurry pre-consolidated under the final pressures of 100 and 400 kPa respectively. The average effective unit weight ($\gamma'$) of the soft and stiff clay layers was ~7.2 kN/m$^3$. The thickness of the top soft clay layer and the bottom stiff clay layer was 9.5 m and 4 m respectively (prototype scale). A ball penetrometer test was also conducted in parallel to the CPT for comparison. The miniature ball penetrometer was 9 mm in diameter. More information on the recently developed ball penetrometer can be found in Watson et al. (1998) and Newson et al. (1999). Ball penetrometer test was conducted at 1.25 mm/s and CPT at 1.15 mm/s to target an identical normalised velocity (nulling relative strain rate effect) and to ensure undrained conditions in the kaolin clay (Finnie & Randolph, 1994).

4.3 Results and discussion

The following section discusses the calibration factors for the cone tip load cell derived from the axial load test at 1g and the ambient pressure test at 150g. The CPT data from
the test in the layered clay are then interpreted using these calibration factors. The
interpreted soil undrained shear strength from the CPT data is compared with that from
the ball penetrometer test data.

4.3.1 Calibration factor from axial load test

The recorded data from axial load test are shown in Figure 4-5. Applied pressure from
the reference load cell is plotted as a function of voltage from the cone tip load cell. A
perfectly linear response can be seen in both loading and unloading stage, with a
negligible divergence between loading and unloading stage and minimal zero-load
error (<0.1% full scale output). A linear fitting provides the calibration factor \( C_{ax, \text{ax}} \) stands for axial load) of 5184.4.

4.3.2 Processing of CPT data based on axial load calibration factor

The obtained calibration factor from axial load test was used to interpret the cone
penetration test data in the layered clay. The soil undrained shear strength was
interpreted according to Lunne et al. (2011)

\[
\begin{align*}
\sigma_{u(CPT)} &= \frac{q_{net}}{N_c} \\
q_{net} &= q - \sigma_{vo}
\end{align*}
\]

where \( \sigma_{u(CPT)} \) is the undrained shear strength of clay from CPT, \( q_{net} \) is the net resistance
from the cone tip, \( q \) is the total measured resistance from the cone tip, \( \sigma_{vo} (= \gamma \times d, \gamma \) is
the total unit weight of the soil and \( d \) is the penetration depth of the cone shoulder) is
the overburden pressure at corresponding depth. The bearing capacity factor of
resistance factor of cone in clay, \( N_c \), was taken as 13.56 (Low et al., 2010).

The result from the ball penetrometer test can be processed to obtain the undrained
shear strength as
Ambient pressure calibration for cone penetrometer test: necessary?

\[ s_{u(ball)} = \frac{q}{N_b} \]  

(4-3)

where the ball bearing capacity factor \( N_b \) was taken as 11.17 (Randolph et al., 2000; Zhou et al., 2013). As the ball penetrometer penetration generally mobilises a full flow-round mechanism around the ball except for the shaft connection area, the soil overburden pressure has a minimal influence on the measured resistance (Randolph, 2004).

The interpreted undrained shear strength profiles from the cone \( s_u(CPT) \) and ball \( s_u(ball) \) penetrometer tests in soft-over-stiff clay (with soft and stiff layer marked with light and dark shaded column separately) are shown in Figure 4-6. It is interesting to notice that a negative undrained shear strength profile between 4.5~9.6 m was obtained from the cone penetration data while an expected positive strength profile was obtained for the full penetration depths from the ball penetrometer data. The negative undrained shear strength in Figure 4-6 is resulted from the negative resistance calculated by Equation 4-2. A similar phenomenon of negative resistance was reported by Boylan & Long (2006) from a field test on peat. The negative undrained shear strength is clearly illogical, leading to further investigation on the cone calibration factor.

4.3.3 Processing of CPT data using both axial load calibration factor and hydraulic pressure calibration factor

The result from hydraulic pressure calibration is shown in Figure 4-7, plotting the ambient hydraulic pressure as a function of voltage response from the cone tip load cell. Surprisingly, the voltage-pressure response (Figure 4-7) shows a negative correlation between the voltage output under ambient pressure, which is the opposite to the one observed from the axial load test (see Figure 4-5). From the fitting curve in Figure 4-7, the calibration factor under ambient pressure \( C_{am} \), ‘am’ stands for ambient
load) is \(-18665 \text{kPa/V}\). It is worthwhile to note that, although the load range of the hydraulic test (323 kPa) is much less than that of the axial load test (2863 kPa), the linear response indicates a constant calibration factor. In the tested clay deposit, the maximum overburden pressure on the cone tip of 206 kPa also lies in this range.

To apply the cone tip load cell calibration factors in the CPT data interpretation, it should be noted that the measured cone tip resistance includes two parts: (1) the cone tip resistance from soil shearing during cone penetration; and (2) the cone tip resistance from the overburden pressure from the soil (see Equation 4-2). The recorded total voltage output can be seen as the contribution of these two components. Therefore, it can be derived from Equations 4-1 and 4-2 that

\[
v_t = \frac{q_{net}}{C_{ax}} + \frac{\sigma_v}{C_{am}}
\]

where \(v_t\) is the total voltage output. It should be noted that, if \(C_{ax} = C_{am}\), the data interpretation becomes easier as \(v_t = q/C_{ax}\) (or \(q = C_{ax} \times v_t\)). In this case, it is apparent that Equation 4-5 degenerates to Equation 4-1, which was used in the CPT data interpretation for Figure 4-6. However, in this case, different calibration factors were obtained from the axial load test and ambient pressure test. Thus, \(q_{net}\) should be calculated from Equation 4-5

\[
q_{net} = (v_t - \frac{\sigma_v}{C_{am}}) \times C_{ax}
\]

With the modified net tip resistance from Equation 4-5, a new strength profile can be interpreted. In this way, the undrained shear strength is modified and compared with the ball penetrometer result. Figure 4-8 shows that the modified net tip resistance has produced a soil strength profile of the layered clay that is very close to that from the ball penetrometer test. This confirms that the modified net tip resistance can provide
the logical interpretation of soil undrained shear strength. This exercise shows that failing to consider the difference in calibration factors for the two components may lead to interpretation error and even illogical result such as negative strength.

However, it is also worth noting that not all cone tip load cells show two different calibration factors under axial load and ambient pressure. Another group of parallel tests were conducted on the cone tip load cells, with the same size and the same type of cone (i.e. 10 mm in diameter and only equipped with tip load cell). Figure 4-9 depicts the results from both calibration tests. It can be seen that both calibration factors are consistent, with a difference being < 8%. This difference of the calibration factors from 1g axial load test and centrifuge hydraulic test might be due to the Poisson’s strain in the load cell under ambient pressure and the increased self-weight of the cone tip under enhanced gravity (150g) (Chow et al., 2017). Since the cone resistance profile was also measured under the influence of enhanced gravity, it is reasonable to adopt the calibration factor from centrifuge hydraulic test for subsequent interpretation. In this case, the negative net resistance would not happen because the overburden pressure would increase the output voltage in a similar ratio, so deducting it from the total tip resistance would not lead to a negative net resistance. Yet, the interpreted strength may not be correct, as explained previously, emphasising the necessity of the ambient pressure test.

This study shows the importance of calibration under ambient pressure because calibration under pure axial pressure cannot always guarantee the load cell behaves in the same way under ambient pressure. Calibration of the tip load cell under atmospheric condition, i.e. pressurised chamber test, has been reported by Peuchen & Terwindt (2014), which is able to reflect the ambient pressure effect on the load cell rather than axial load test. However, the pressurised chamber test is not suggested by
the standards for cone tip load cell (ASTM, 2012), and it is not commonly adopted in in-house calibration check of calibration factor. In fact, the pressurised triaxial apparatus are normally used to check for pore pressure transducer as well as net area ratio of cones (Mayne, 2007). In addition and critically, the hydraulic pressure test introduced in this chapter can also examine the load cell behaviour under water e.g. potential water ingress and corresponding drifting/oscillation of response during actual testing, which could not be simulated by the air pressure chamber test.

4.4 Summary and concluding remarks

The total cone resistance measured by CPT is used to interpret soil undrained shear strength. The total cone resistance includes two parts: the shear resistance and the overburden pressure. The cone tip load cell needs to be calibrated using the axial load test (for the shear resistance) and the ambient pressure test (for the overburden pressure). When the calibration factors under the axial load test and under the ambient pressure test are the same (or similar), the interpretation can be straightforward. However, when the two calibration factors are different, both calibration should be used for interpreting the CPT data. The hydraulic test in centrifuge has been proved to work well to produce the cone tip load cell calibration factor under ambient pressure.

4.5 References

Ambient pressure calibration for cone penetrometer test: necessary?


Ambient pressure calibration for cone penetrometer test: necessary?

Figure 4-1. Cone penetrometer: (a) schematic diagram; (b) cone penetrometer used in centrifuge tests

Figure 4-2. Cone penetrometer subjected to both ambient pressure and axial resistance when penetrating in soil
Figure 4-3. Axial load test at 1g

Figure 4-4. Hydraulic pressure test in a centrifuge
Ambient pressure calibration for cone penetrometer test: necessary?

Figure 4-5. Stress-voltage response of cone tip load cell under axial load

\[ C_{ax} = \frac{\Delta q}{\Delta v} = 5184.4 \]

Figure 4-6. Comparison of undrained shear strength of layered clay from ball penetrometer test and cone penetrometer test (using calibration factor under axial load test)
Figure 4-7. Stress-voltage response of cone tip load cell under ambient pressure from centrifuge hydraulic calibration test (150g)

\[ C_{am} = \frac{\Delta q}{\Delta v} = -18665 \]

Figure 4-8. Modified shear strength from cone penetrometer test using both calibration factors under the axial load test and under the ambient pressure test and its comparison with ball penetrometer result
Figure 4-9. Axial load calibration test result and centrifuge hydraulic calibration test for another cone tip load cell.
CHAPTER 5  EFFECT OF TRAPPED CAVITY MECHANISM ON T-BAR PENETRATION RESISTANCE IN UNIFORM CLAYS

ABSTRACT

This chapter describes large deformation finite element (LDFE) analysis of the penetration of the T-bar penetrometer in uniform clays, identifying soil flow mechanisms around the T-bar, the extent of any cavity above the T-bar and the evolving penetration resistance profile. A trapped cavity above the advancing T-bar penetrometer and its influence on the corresponding bearing capacity factor are the crucial findings of this chapter. With the progress of T-bar penetration, a shallow failure mechanism gradually turns to a flow-round mechanism. However, the soils backflow around both sides of the T-bar meets at a distance above the top surface of the T-bar, leaving a trapped cavity above the penetrating T-bar. Finally, the trapped cavity is being backfilled and a fully localised flow-round mechanism forms. The formation and evolvement of the trapped cavity mechanism has been studied extensively exploring a large range of soil normalised undrained shear strength and T-bar surface roughness. It is shown that the depths of forming a trapped cavity and being fully filled with soil would increase with normalised undrained shear strength and T-bar roughness. The duration, i.e. depth span, of trapped cavity mechanism from forming to end also increases with undrained shear strength. The presence of a trapped cavity is shown to reduce the T-bar bearing capacity factor that is commonly used in practice. According to the depth span of trapped cavity stage, there are three scenarios:
(i) for very soft soils \( (s_u/\gamma'D \leq 1) \), shallow failure mechanism is directly followed by full-flow mechanism since the trapped cavity stage is negligible; (ii) for soils with \( 1 < s_u/\gamma'D \leq 8.3 \), all three stages—shallow failure mechanism, trapped cavity mechanism and full-flow mechanism—can be observed; and (iii) for soil with \( s_u/\gamma'D > 8.3 \), the trapped cavity is not fully closed up to \( 30D \), leading to a lower stabilised resistance factor on the profile compared to the other two scenarios. A systematic interpretation procedure is therefore proposed to account for the effect of the trapped cavity for more accurate interpretation of soil undrained shear strength from the T-bar penetration resistance.

### 5.1 Introduction

Site investigation tools play a key role in deep water exploration due to the high cost and difficulty in extracting soil samples from the offshore sites for laboratory testing. Site investigation tools include shear vane, cone penetrometer and the newly developed ‘full-flow’ penetrometers (i.e. T-bar or ball penetrometers) (Figure 5-1). T-bar penetrometer was first implemented by Stewart & Randolph (1994), and identified as advantageous for investigating deep water fine-grained sediments due to: (i) providing a continuous resistance profile that can be directly interpreted to corresponding soil strength profile; (ii) the large projection area ratio (5–10 times larger than the cone penetrometer) leading to high resolution in soft seabed resistance; and (iii) the negligible influence of overburden pressure to the total resistance (Lu, 2004; Zhou & Randolph, 2009), eliminating the necessity of overburden pressure corrections needed for the cone penetrometer. Moreover, the miniature version of the T-bar penetrometer is now commonly used for characterising fine-grained soil samples in both centrifuge and 1g tests for soil characterisation (e.g. Purwana et al., 2005;
Effect of trapped cavity mechanism on T-bar penetration resistance in uniform clays (Hossain et al., 2011). Therefore, accurate interpretation of the resistance profile from T-bar tests is of great importance to the geotechnical community.

The foundation of interpretation method was laid by Randolph & Houlsby (1984), simplifying the problem as a cylinder deeply penetrated in the soil. With the assumed fully localised flow-round failure (Figure 5-2a), they found that soil undrained shear strength ($s_u$) is in linear relationship with resistance (Equation 5-1) through a resistance factor ($N_t$). Later, improved plasticity solutions were provided by Martin & Randolph (2006). The resistance factor ranges from 9.1~11.9 depending on the T-bar roughness factor, and 10.5 is recommended and commonly adopted in centrifuge tests.

$$s_u = \frac{q_{net}}{N_t} \quad (5-1)$$

To provide guidance for the interpretation of soil strength at shallow depths associated with a shallow failure mechanism with soil flow towards the surface maintaining an open cavity above the T-bar, several analytical studies have been conducted (Murff et al., 1989; Aubeny et al., 2005). A cylindrical bar was pre-embedded at different shallow depths (up to 5 times the T-bar diameter, $D$) and an open cavity was considered in the wake of the bar (Figure 5-2b and c). It is found that the T-bar resistance factor, $N_t$, for shallow embedment depths can be 30% lower than that for the deep embedment with fully localised flow-round failure. The interpretation of soil strength at shallow depths is particularly critical for the design of shallow foundations, mudmats and pipelines in deep waters.

The development of large deformation finite element (LDFE) method enables the evolvement of soil flow mechanisms and resistance factors during continuous penetration of the T-bar to be studied systematically. White et al. (2010) explored T-
bar penetration in uniform clay. Two failure mechanisms were identified as (i) shallow failure mechanism with an open cavity above the bar, and (ii) sudden close of the cavity and deep fully localised flow-round failure mechanism. The depth of attaining the latter mechanism was shown to increase with increasing normalised undrained shear strength, $s_u/\gamma'D$ where $\gamma'$ is the submerged unit weight of the soil, and $D$ is the diameter of the T-bar cylinder. The T-bar resistance profile was therefore divided into shallow (before mobilising the flow-round failure) and deep (after mobilising flow-round failure) penetration stages (Figure 5-2d) and corresponding soil strength interpretation framework was proposed.

Tho et al. (2011) however identified a trapped cavity mechanism (Figure 5-2e), with the cavity being closed at a distance above the T-bar rather than just above the T-bar, in between the shallow and deep mechanisms for $s_u/\gamma'D > 3$ and for a smooth (roughness factor $\alpha = 0$) T-bar. Critically, the trapped cavity led a 12% lower T-bar resistance factor compared to the flow-round failure mechanism. This is due to shorter flow path or shear failure plane.

Recently, the soil failure mechanisms during the T-bar penetration in two-layer clays were revealed through centrifuge model tests and subsequent particle image velocimetry (PIV) analysis (Chapter 2 of the thesis). The penetration in the very thick top layers relative to the T-bar diameter can represent penetration in uniform layers. Interestingly, a trapped cavity mechanism was observed for $s_u/\gamma'D = 5.24$, and no trapped cavity (or may be closed very shortly and hence not evident—can be neglected) for $s_u/\gamma'D = 1.42$.

For the standard T-bar penetrometer commonly used in offshore site investigations ($D = 0.04$ m) and commonly encountered offshore clay strengths ($s_u = 5$–$25$ kPa), the
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range of normalised strength falls between 20~100. The soil strength interpretation
framework proposed by White et al. (2010) did not take into account the trapped cavity,
and limited to $s_u/\gamma'D \leq 10$. The framework proposed by Tho et al. (2011) considered
the trapped cavity but it is also limited to $s_u/\gamma'D \leq 10$, fully smooth T-bar (roughness
factor $\alpha = 0$) and up to $10D$ in penetration depth. Thus it is imperative to conduct a
more systematic study to cover the practical range of $s_u/\gamma'D$ and T-bar roughness factor.
This chapter describes the results from LDFE analyses to provide insight into the
evolving soil failure mechanisms in both soft and stiff clays, and corresponding
influence on penetration resistance profiles. Both centrifuge scale T-bar and field scale
T-bar were considered, covering a rather wide range of $s_u/\gamma'D = 1$~100. The influence
of T-bar roughness factor was also investigated. Based on the results, a systematic
interpretation procedure was established for offshore designs.

5.2 Numerical analysis

This study has simplified the T-bar as an infinite long cylindrical bar, thus the T-bar
penetration was simulated as a plane strain problem, which is a commonly adopted
assumption (Lu et al., 2000; White et al., 2010). The T-bar was penetrated
continuously from the soil surface. The diameter of the T-bar, $D = 0.04$ m was
considered for simulating the field scale T-bar used in site investigation, and $D = 0.75$
m for simulating the prototype scale T-bar of a model miniature T-bar (i.e. $D = 0.005$
m under 150 g-level) commonly used in centrifuge testing. Due to the symmetry of
the problem, only half of the T-bar and the soil domain was modelled. The size of the
domain was set as $50D$ in both width and height for the field-scale T-bar and $32D$ for
the centrifuge-scale T-bar, which were large enough to eliminate boundary effect.
Soil was considered as elastic-perfect plastic material obeying a Tresca failure criterion, with the yield stress solely determined by the undrained shear stress. The elastic behaviour was defined by Poisson’s ratio of 0.49 and Young’s modules $E = 500s_u$ throughout the soil profile. A uniform submerged unit weight of $\gamma' = 6$ kN/m$^3$ was adopted over the soil depth, representing a typical average value encountered in the field. Uniform undrained shear strength profiles with $s_u = 2\sim31.5$ kPa were considered, covering the most commonly encountered clay strength of hydrocarbon active seabed sediments (Menzies & Roper, 2008). Therefore, the investigated normalised strength $s_u/\gamma' D$ ranged from 1 to 100. Table 5-1 lists a summary of the considered parameters.

The interface between the T-bar and soil was modelled using elastoplastic nodal joint elements (Herrmann, 1978), and the soil strength at the soil-T-bar interface was modelled as $\alpha s_u$, where $\alpha$ is the roughness factor, as defined by Randolph & Houlsby (1984). For this study, an intermediate roughness condition ($\alpha = 0.3$) was adopted for the majority of simulations, which is accepted widely for both field and miniature T-bars. However, the effect of roughness on the depth span of the trapped cavity mechanism and resistance factor was also examined considering $\alpha = 0\sim0.7$ (Groups VI, VII and VIII; Table 5-1).

In this study, LDFE analysis were conducted using the FE package AFENA (Carter & Balaam, 1995) developed at the University of Sydney coupled with RITSS (remeshing and interpolation technique with small strain) by Hu & Randolph (1998a) for remeshing and interpolation. $H$-adaptive mesh refinement cycles (Hu & Randolph, 1998b) were implemented to optimise the mesh, hence to minimise discretisation errors, especially in highly stressed zones. This method falls in the category of ALE (arbitrary Lagrangian Eulerian) method (Ghosh & Kikuchi, 1991), whereby a series
Effect of trapped cavity mechanism on T-bar penetration resistance in uniform clays of small-strain analysis increments are followed by remeshing, and then interpolating the field quantities (stress and material properties) between the Gauss points in the new mesh and those in the old mesh. The LDFE/RITSS method has been proven its robustness in various geotechnical boundary value problems (Lu et al., 2001; Ma et al., 2015; Zhou et al., 2013). Penetration of the T-bar is simulated from the mudline and by specifying incremental displacements. The displacement incremental size and number of steps of small strain analysis between each remeshing stage were chosen such that the cumulative penetration in each remeshing stage remained in the small strain range and less than half of the minimum element size. Six-node triangle elements with three internal Gauss points were adopted for all analyses.

5.3 Validation of numerical model

In this section, the FE model was validated against two groups of existing results: (a) the analytical solutions of the pre-embedded and fully embedded (no cavity) T-bar; (b) the centrifuge test results of continuously penetrated T-bar. Both T-bar resistance factors and soil flow mechanisms around the T-bar were studied.

5.3.1 Validation against analytical solutions

The T-bar penetration resistance from the LDFE analysis is compared with previous analytical solutions (Randolph & Houlsby, 1984; Martin & Randolph, 2006; Group I in Table 5-1). The T-bar was pre-embedded to $10D$ under the seabed surface, in order to eliminate the surface influence and to invoke the classical flow-round mechanism around the T-bar (Lu, 2004). The roughness factor was varied from fully smooth (i.e. $\alpha = 0$) to fully rough (i.e. $\alpha = 1$). The comparison between the current FE results and the upper and lower bounds solutions is shown in Figure 5-3. The resistance factor, $N_t$, from the current analysis is obtained by rearranging Equation 5-1 (i.e. $N_t = q_{ue}/s_u =$...
\( F_{\text{net}} / (A_s) \), where the T-bar net resistance, \( F_{\text{net}} \), is the measured force deducting the buoyancy force \( (F_b) \), and \( A \) is the projection area of the T-bar. For the pre-embedded T-bar, buoyancy force equals to \( \gamma' A_s \), where \( A_s \) is the submerged cross-sectional area of the T-bar, i.e \( \pi D^2 / 4 \). For the plain strain problems, the projected area of the T-bar equals to the diameter multiplied by unit length, so the equation can be expressed as

\[
N_t = \frac{F - \gamma' A_s}{D s_u}
\]  

(5-2)

It can be seen that the results from the current LDFE model lie slightly above the exact solution with the maximum difference being 3.7%. This over estimation can be due to the element size used in the LDFE analyses. This difference was thought to be acceptable and hence the corresponding minimum element size of \( h_{\text{min}} = 0.022D \) was used for all other analyses. The reasonable agreement between the numerical results and the exact solutions confirms the accuracy of the current LDFE model. It is also worth to note that although only the field-scale T-bar was adopted for this validation, corresponding sensitivity analyses confirmed that the T-bar dimensions have no influence on the fully embedded T-bar resistance factor.

### 5.3.2 Validation against centrifuge test results

Validation in terms of soil failure mechanisms were carried out against centrifuge observation reported in Chapter 2. A T-bar penetration test was conducted in a two-layer stiff-soft clay deposit (TB2; Table 2-1, Chapter 2): (i) the top layer of \( s_{u1} = 29.1 \) kPa, \( h_1 = 9.295 \text{ m} \) (\( h_1/D = 12.4 \)); (ii) the bottom layer of \( s_{u2} = 7.5 \) kPa and \( h_2 = 4.205 \text{ m} \) (\( h_2/D = 5.6 \)). The images of soil failure mechanisms were captured by a camera and subsequently analysed by particle image velocimetry (PIV) analysis. An LDFE analysis was carried out considering the identical parameters of centrifuge testing and a T-bar roughness of \( \alpha = 0.3 \) (Group II, Table 5-1).
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The soil failure mechanisms from the LDFE analyses were compared with the PIV results for the top layer in Figure 5-4 (noting that the depth \( d \) is from the soil surface to the invert of the T-bar; Figure 5-1). The shallow failure mechanism with the soil deformation directs towards the surface (Figure 5-4a), and soil flow around the T-bar with a trapped cavity above the T-bar (Figure 5-4b, c); all captured reasonably by the LDFE analysis. Note, the trapped cavity remained open even at \( d/D = 9.8 \) (Figure 5-4c) for this case with \( su/\gamma'D = 5.24 \).

It is clear from these two validation exercise that the LDFE/RITSS model can capture both penetration resistance and soil failure mechanisms with reasonable accuracy, which provides confidence to use the numerical method for the parametric analyses. The expected agreement is reached between PIV analysis and numerical analysis on the features of forming a trapped cavity and decreasing cavity size with penetration down. However, it is noticed that a wider cavity in PIV analysis, while a longer and narrower cavity simulated in LDFE analysis. One possible reason is that, in physical modelling, the shrinkage of the inner cavity could be obstructed by water trapped inside the cavity. Another possible reason is the adopted isotropic elastic perfect plastic Tresca model could not consider anisotropic strength in compression and tension as well as softening behaviour exhibited by soft clay. These might cause the difference in backflow and cavity shape.

5.4 Parametric study-results and discussions

5.4.1 Evolvement of soil flow mechanism and trapped cavity mechanism

To illustrate typical evolution of soil failure mechanisms with the advancement of a T-bar penetration, a case with centrifuge-scale T-bar (i.e. \( D = 0.75 \, \text{m} \)) and \( su/\gamma'D = 3 \) was selected (Group III, Table 5-1). The mobilised mechanisms at various depths from
LDFE/RITSS analysis are displayed in Figure 5-5. At a very shallow penetration depth of $d/D = 0.5$ (Figure 5-5a), the soil flow directs towards the surface, leading to surface heave adjacent to the T-bar. A cavity is formed at the wake of the T-bar. At $d/D = 1.5$ (Figure 5-5b), with the increase of overburden pressure, the soil flow direction changes from towards the surface to flow back towards the open cavity. With further penetration of $d/D = 4.4$ (Figure 5-5c), the soil backflow continues and the cavity above the T-bar becomes significantly narrow.

At a deep penetration of $d/D = 5.9$ (Figure 5-5d), the backfilled soil completely touched the vertical centreline axis at a distance of ~1.6$D$ above the T-bar i.e. a ‘trapped cavity’ is formed. This is termed as trapped cavity as there is the open cavity still standing above the backfilled closed soil and there is no connection between these two cavities (Figure 5-5d). With further penetration, the soil backflow forces the trapped cavity to become shorter and narrower (see $d/D = 10.1$ in Figure 5-5e). The trapped cavity eventually disappears at $d/D = 10.5$ and a fully localised flow-round mechanism mobilised (Figure 5-5f). In this case, the transition from the formation of a trapped cavity to the mobilisation of the flow-round mechanism takes ~5$D$ penetration (Figure 5-5d~f). In the following parametric study, this transition period with the existence of a trapped cavity needs to be identified and the corresponding influence on the T-bar penetration resistance are quantified for more accurate interpretation of shear strength.

Based on the above observations, the soil flow mechanisms around a T-bar during its continuous penetration can be divided to three stages: stage I - shallow failure mechanism, where the soil flows towards the surface, an open cavity is formed above the T-bar, and the soil flow is influence by the overburden pressure; stage II - trapped cavity mechanism, where, along with an open cavity close to the soil surface, a trapped
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cavity is formed above the T-bar; stage III - deep fully localised flow-round failure
mechanism, where there is no trapped cavity above the T-bar and soil flows around
full T-bar periphery.

5.4.2 Effect of $s_u/\gamma'D$ on the duration of trapped cavity mechanism (Stage II)

A series of parametric study (Groups IV & V; Table 5-1) was conducted to study the
effect of $s_u/\gamma'D$ on the depth span of existing trapped cavity mechanism for both
centrifuge- and field-scale T-bars. For clarity, the transition depth from stage I to stage
II, $h_t$ (i.e. the depth of forming the trapped cavity) was identified when the cavity walls
at the top of the trapped cavity come closer than the criteria, i.e. numerically the
minimum distance between the cavity wall and the centreline vertical axis is $\leq 0.1D$
due to the soil backflow. The end of Stage II, $h_f$, was defined as the depth where
trapped cavity closes up, i.e. numerically the maximum width/height of the trapped
cavity is $\leq 0.01D$ (this is also the beginning of Stage III – deep flow-round/full flow
mechanism). The start and end depths correspond well with the kinks on the resistance
factor profiles, which will be demonstrated in the next section.

The values of $h_t$ and $h_f$ are plotted in Figure 5-6 as a function of $s_u/\gamma'D$ in a semi-
logarithmic scale. The values of $h_t$ and $h_f$ from Tho et al. (2011) are also included in
Figure 5-6 for comparison. White et al. (2010) did not recognise the trapped cavity
mechanism rather proposed an expression for the depth of attaining (or depth of
transitioning from shallow to) deep flow-round mechanism, and the corresponding
curve is also plotted in Figure 5-6. As noted previously, White et al. (2010) and Tho
et al. (2011) have covered a relatively small range of normalised strength ($s_u/\gamma'D =
0.1\sim10$), which is mainly applicable to centrifuge scale T-bar and offshore pipelines,
while this study encompassed a much wider range of $s_u/\gamma'D = 1\sim100$ including in-situ
T-bars.
Overall the results from this study are in good agreement with the existing solutions.

Over the range of $s_u/\gamma'D = 0.1\sim1$, the trapped cavity mechanism can be ignored as $h_t \approx h_f$. For $s_u/\gamma'D > 1$, the gap between $h_t$ and $h_f$ increases dramatically with increasing $s_u/\gamma'D$. The start of the trapped cavity mechanism, $h_t$, increases gradually from 2.4 to 9.7 for $s_u/\gamma'D = 1\sim16.8$, and then stabilises at $h_t/D \approx 9.7$ for $s_u/\gamma'D > 16.8$, which can be expressed as

$$
\frac{h_t}{D} = \begin{cases} 
2.58 \left( \frac{s_u}{\gamma'D} \right)^{0.46} + 0.24 \left( \frac{s_u}{\gamma'D} \right)^{-0.63} & \text{for } \frac{s_u}{\gamma'D} \leq 16.8 \\
9.7 & \text{for } \frac{s_u}{\gamma'D} > 16.8
\end{cases}
$$

(5-3)

This means that, for stronger soils of $s_u/\gamma'D > 16.8$, the trapped cavity starts to form at the same penetration of $h_t/D = 9.7$ regardless of the increase in soil strength. For example, for a common offshore clay with $\gamma' = 6$ kN/m$^3$ and a standard in-situ T-bar with $D = 0.04$ m, $s_u/\gamma'D > 16.8$ means $s_u > 4.0$ kPa. In other words, in offshore site investigations, for a uniform clay with strength of $s_u > 4$ kPa, a trapped cavity is expected to form after the penetration reaches 0.4 m (as $h_t/D = 9.7$).

Following the formation of a trapped cavity, all T-bar penetration analyses were carried out up to $d/D = 25$ (Groups III, IV and V in Table 5-1). The closure of the trapped cavity was only observed for $s_u/\gamma'D \leq 8.3$. The values of $h_f$ increase exponentially with $s_u/\gamma'D$. For $s_u/\gamma'D > 8.3$, the cavity did not close i.e. the depth of $h_f$ was not reached within $d/D = 25$. Best fitting through the obtained results, a design formula for $h_f$ can be expressed as

$$
\frac{h_f}{D} = 3.51 \left( \frac{s_u}{\gamma'D} \right)^{0.98} \text{ for } \frac{s_u}{\gamma'D} \leq 8.3
$$

(5-4)
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For $s_d/\gamma'D > 8.3$ (i.e. $s_u > 2.0$ kPa for a standard in-situ T-bar with $D = 0.04$ m), intuitively the cavity closure will occur down to a deeper penetration depth with further penetration. It can be assumed that 1st degree calculation for $h_f$ can be calculated by extrapolating Equation 5-4. Regardless, this greater depth span of existing trapped cavity means a stronger (or longer) influence on the T-bar penetration resistance and interpreting shear strength.

5.4.3 Effect of trapped cavity mechanism on T-bar resistance factor

Based on the numerical results in Figure 5-6 with the T-bar penetration up to $d/D = 25$, there are three scenarios: scenario 1 – for very soft clays with $s_d/\gamma'D \leq 1$, the T-bar penetration experiences stages I and III of soil failure mechanisms without any trapped cavity mechanism (i.e. stage II); scenario 2 - for soft clays with $1<s_d/\gamma'D \leq 8.3$, the T-bar penetration experiences all three stages of soil failure mechanisms (i.e. stages I, II and III); scenario 3 – for stiff clays with $s_d/\gamma'D > 8.3$, the T-bar penetration could not mobilise full-flow mechanism (stage III) for up to $25D$ (i.e. only experience stages I and II). Three typical profiles resistance factor, $N_t$, have been selected from these three range of $s_d/\gamma'D$ and are displayed in Figure 5-7. Both critical depths $h_i$ and $h_f$ are marked on the profiles. Prediction using White et al.’s (2010) expression is also included in Figure 5-7 for comparison.

To calculate $N_t$, the T-bar buoyancy force needs to be deducted from the measured resistance to obtain net T-bar resistance. In contrast to the pre-embedded case, the buoyancy force needs to be adjusted by a multiplier, i.e. $f_b\gamma'A_s$, to account for the influence of enhanced buoyancy effect due to the upheave of the soil adjacent to the pipe (Merifield et al., 2009; White et al., 2010; Tho et al., 2011). The buoyancy correction multiplier was taken as 1.3 at shallow embedment, which is in consistent
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with that proposed by Tho et al. (2011) and Merifield et al. (2009). For submerged area $A_s$ with penetration depth ($d$) up to $0.5D$

$$A_s = \frac{D^2}{4} \left[ \sin^{-1} \sqrt{4d(D-d)} - \frac{2}{D} \sqrt{d(D-d)} \right] - 2 \left( 1 - \frac{2d}{D} \right) \sqrt{d(D-d)}$$

for $\frac{d}{D} \leq 0.5$ (5-5)

After $0.5D$, the submerged area gradually increases with the progress of backflow. Referring to Figure 5-5, $A_s$ increase to 82% of the maximum $A_s (=\pi D^2/4)$ at $1.5D$, 99% at $4.4D (~h_t)$ and 100% at $10.5D (~h_f)$. Although the pattern may vary depending on $s_u/\gamma'D$, for continuity and simplicity, the buoyancy force ($F_b$) was set to be linearly increasing from the $d = 0.5D$ to $d = h_t$, with the limiting value of $\gamma'\pi D^2/4$.

$$F_b = \begin{cases} 
1.3\gamma'A_s & \text{for } d \leq 0.5D \\
\frac{1.3\gamma'D^2}{8} + \frac{0.0875\gamma'\pi D^2}{h_f - 0.5D} (h_t - d) & \text{for } 0.5D < d < h_t \\
\frac{\gamma'\pi D^2}{4} & \text{for } d \geq h_t 
\end{cases}$$

(5-6)

It is worth noting that the influence of buoyancy can be reduced with increased $s_u/\gamma'D$ because the contribution of buoyancy becomes less significantly compared to the contribution from $s_u$ term (Tho et al., 2011).

An example resistance factor profile of scenario 1 is shown in Figure 5-7a. For $s_u/\gamma'D = 0.67$, it shows that $N_t$ increases sharply up to $d = h_t \approx h_t$ in stage I, and then attains to the limiting value of $N_{ff} = 10.61$ (for the T-bar with $\alpha = 0.3$) in stage III. The expression in White et al. (2010) underestimates $N_t$ in stage I.

The example resistance factor profile of scenario 2 and 3 are shown in Figure 5-7b. For scenario 2 ($s_u/\gamma'D = 3$), it shows that $N_t$ rises rapidly up to $d = h_t$ in stage I, and then increases slowly up to $d = h_t$ in stage II before attaining the same stabilised value for scenario 1, i.e. $N_{ff} = 10.61$ at $h_t = 10.53D$. The limiting resistance factor $N_{ff} = 10.61$
Effect of trapped cavity mechanism on T-bar penetration resistance in uniform clays agrees well with analytical solutions \((N_t = 10.22\sim10.69\) for \(\alpha = 0.3\) by Randolph & Houlsby, 1984; Martin & Randolph, 2006) and pre-embedded LDFE analysis result \((N_t = 10.61\) for \(\alpha = 0.3\)). For scenario 3 \((s_u/y'D = 41.6)\), the T-bar penetration depth of 30\(D\) is not sufficient to reach \(h_t\). However, from the similar trend of increasing tendency of \(N_t\) in stage II, it can be concluded that the profile may reach to the limiting value down to a very deep penetration depth. Extrapolating Equation 5-4 that depth can be \(h_t/D = 146\). Based on the above study, a typical schematic T-bar resistance factor profile including all three stages is shown in Figure 5-8, with T-bar resistance factor defined for different stages. In stage I, \(N_{sf}\) (‘sf’ stands for shallow failure) rises sharply with depth and reaches \(N_{cf}\) (‘cf’ stands for cavity formed) at \(d = h_t\). In stage II, \(N_{tc}\) (‘tc’ stands for trapped cavity) increases gradually and linearly with depth from \(N_{cf}\) and attains to \(N_{ff}\) (‘ff’ stands for full-flow) at \(d = h_t\). In stage III, \(N_t\) remains constant with the value of \(N_{ff}\). The resistance factor associated with the three stages will be quantified accordingly.

It can be seen that for all the three scenarios, the resistance factor \(N_{sf}\) is either under or overestimated by White et al.’s (2010) expression (Figure 5-7a~b) because it was fitted from profiles without considering trapped cavity mechanism. Therefore, a new prediction formula for \(N_{sf}\) is required. Following the development procedure by White et al. (2010), the resistance factor \((N_{sf})\) and penetration depth \((d/D)\) from the analyses of Groups III, IV and V (Table 5-1) are normalised by \(N_{ff}\) and \(h_tD\) for \(s_u/y'D \leq 1.0\) and \(N_{cf}\) and \(h_tD\) for \(s_u/y'D \geq 1.0\). The normalised resistance profiles are shown in Figure 5-9 and can be expressed as
The resistance factor at the start of trapped cavity mechanism \((d = h_t)\), which is referred to as \(N_{ct}\), is around 8% lower than that at \(d = h_f\) i.e. \(N_{ff}\) for both cases in Figure 5-7b. It also shows that \(N_{tc}\) increases linearly from \(N_{ct}\) to \(N_{ff}\) for \(s_u/\gamma'D \leq 8.3\) and stays at \(N_{ct}\) mostly for \(s_u/\gamma'D > 8.3\). Hence the resistance factor \(N_{tc}\) in stage II can be presented as

\[
N_{sf} = \begin{cases} 
-0.6 \frac{s_u}{\gamma'D} + \left( N_{ff} + 0.6 \frac{s_u}{\gamma'D} \right) \left( \frac{d}{h_t} \right)^p & \text{for } \frac{s_u}{\gamma'D} < 1 \\
-0.6 \frac{s_u}{\gamma'D} + \left( N_{cf} + 0.6 \frac{s_u}{\gamma'D} \right) \left( \frac{d}{h_t} \right)^p & \text{for } 1 \leq \frac{s_u}{\gamma'D} \leq 8.3 \\
-4.7 + \left( N_{cf} + 4.7 \right) \left( \frac{d}{h_t} \right)^{0.12} & \text{for } \frac{s_u}{\gamma'D} > 8.3
\end{cases}
\]  

\[
p = 0.33 \left( \frac{s_u}{\gamma'D} \right)^{-0.43}
\]  

White et al.’s. (2010) prediction curve overestimates resistance factor as high as 8%, and critically, without consideration of a trapped cavity, shows a different trend for stage II with earlier attainment to \(N_{ff}\).

### 5.4.4 Effect of roughness factor \((\alpha)\)

The above investigations are based on \(\alpha = 0.3\). To investigate the effect of \(\alpha\) on the trapped cavity mechanism, formation and span of the trapped cavity, and penetration resistance profile, further analyses have been carried out varying \(\alpha = 0 \sim 0.7\) for both centrifuge- and field-scale T-bars (Groups VI, VII and VIII). Typical resistance factor profiles for \(\alpha = 0 \sim 0.7\) but with \(s_u/\gamma'D = 41.6\) (Group VI) are plotted in Figure 5-10.

The depth of attaining a trapped cavity, \(h_t/D\), is marked on the profiles. It is seen that
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$h/D$ increases with increasing $\alpha$ due to delayed soil backflow. To highlight this difference, the values of $h_t/D$ and $h_f/D$ for $\alpha = 0.3$ and 0.7 and $s_u/y'D = 1$~104.2 are plotted together in Figure 5-11 (Groups IV and V vs. Groups VII and VIII). Within the penetration of $d/D = 25$, the trapped cavity did not close with $s_u/y'D > 5$ for $\alpha = 0.7$ and with $s_u/y'D > 8.3$ for $\alpha = 0.3$ (as discussed before). This means the depth of existing trapped cavity is longer for rougher T-bar. It can be seen from Figure 5-11 that the overall trend of both $h_t/D$ and $h_f/D$ for $\alpha = 0.7$ is very similar to that for $\alpha = 0.3$. The difference in both $h_t/D$ and $h_f/D$ between $\alpha = 0.3$ and $\alpha = 0.7$ increases with increasing $s_u/y'D$. In addition, the gap between $h_t/D$ and $h_f/D$ (i.e. the span of stage II) for $\alpha = 0.7$ is much wider than that for $\alpha = 0.3$. In summary, T-bar roughness has significant influence on the trapped cavity mechanism and its depth span of existence, and hence on the interpretation of shear strength.

The profile of $N_t$ for $\alpha = 0.7$ over $s_u/y'D = 1$~104.2 are shown in Figure 5-12 (Groups VII and VIII). The resistance factors for both trapped cavity mechanism and full-flow mechanism ($N_{cf}$ and $N_{ff}$) are marked on the profiles. In general, the form of the $N_t$ profiles in all three stages are very similar to those for $\alpha = 0.3$. The penetration depth of $d/D = 25$ was sufficient for attaining limiting resistance factor $N_{ff}$ for $s_u/y'D = 1$~5, but not for $s_u/y'D = 7$~104.2. For all normalised strengths, the resistance factor at $d = h_t$ i.e. $N_{cf}$ of 10.49 is consistent and is around 13% lower than the limiting resistance factor of $N_{ff} = 11.88$.

The Figure 5-10 and Figure 5-12 clearly show that the values of $N_{cf}$ and $N_{ff}$ for a particular $\alpha$ are somewhat constant regardless of $s_u/y'D$. The values of $N_{cf}$ and $N_{ff}$ for various roughness factors are listed in Table 2-1. The difference of $N_{tc}$ and $N_{ff}$ increases from 6% for $\alpha = 0$ to 8% for $\alpha = 0.3$ and to 13% for $\alpha = 0.7$. As discussed, this is because a rougher T-bar tends to delay the soil backflow and maintains a larger trapped
cavity for deeper depth span, as typically evidenced from Figure 5-13 illustrating the trapped cavity at penetration depth of \( d/D = 16 \) for \( \alpha = 0, 0.3 \) and 0.7.

5.5 Interpretation framework and exercise

Based on the results presented previously, a suggested procedure for estimating undrained shear strength from T-bar penetration resistance in a single layer clay is outlined in Figure 5-14, covering all three scenarios revealed in this chapter. The procedure is based on a more typical (for both field and centrifuge) T-bar roughness of \( \alpha = 0.3 \), though can be modified for other values of \( \alpha \) using corresponding values of \( N_{ct}, N_{ft}, h_t, h_t \) (e.g. for \( \alpha = 0.7 \) using Figure 5-12, Figure 5-13 and Table 5-2).

Although uniform strength was considered, in field applications, the small size of the T-bar means that the normalised local soil strength gradient – expressed in dimensionless for as \( kD/s_u \) – is usually small (White et al., 2010). As such, the procedure will be applicable in soil with varying strength with depth – allowing the local value of \( s_u/\gamma'D \) to be used.

To examine the capability of this framework, a centrifuge test result from a miniature T-bar \((D = 1\,m)\) penetration in single layer kaolin clay deposit (liquid limit, LL = 61%; plastic limit, PL = 27%; \( G_s = 2.6; \) Stewart, 1992), as shown in Figure 2-15, is selected.

Following the procedure, first, the soil undrained strength was estimated using Equation 5-1 with \( N_{ft} = 10.61 \), based on which an initial value of \( s_u = 29 \) kPa was determined, giving \( s_u/\gamma'D = 4.03 \) where \( \gamma' = \sim 7.2 \) kN/m\(^3\). Then \( h_t/D \) and \( h_t/D \) were calculated using Equations 5-3 and 5-4 respectively, and resistance factors for stage I, stage II and stage III, using Equations 5-7 and 5-9 and \( N_{ft} = 10.61 \) respectively.
Effect of trapped cavity mechanism on T-bar penetration resistance in uniform clays

The deduced shear strength profile shows a nearly uniform clay profile except for shallow depth with $d/D < 3$ due to softening of the pre-consolidated sample. This resembles the expected profile for a pre-consolidated clay deposit. In contrast, the deduced strength profile using the plasticity solution with flow-round mechanism, i.e. without considering shallow and trapped cavity mechanisms, shows a gradually increasing trend up to $d/D = 11$. This means the shear strength prior to $d/D < 11$ is significantly underestimated. For the predicted resistance profile by White et al. (2010), it matches the first half of stage I and stage III, however it underestimates the strength for the second half of stage I and stage II. This part may be critical for designing surface and shallow foundations, mudmats and pipelines. The framework by Tho et al., (2011) is only for smooth T-bar with a limiting resistance factor value of 9.14, and as such the corresponding prediction cannot be used directly for comparison.

5.6 Conclusions

The mechanism and resistance factor during the T-bar penetration in uniform clay has been studied through LDFE analysis. A novel trapped cavity mechanism has been revealed and studied extensively by varying normalised soil undrained shear strength in a large range and T-bar roughness factor. The following conclusions can be drawn from this chapter.

- Compared to the conventionally used plasticity solutions with deep flow-round mechanism, shallow failure mechanism and trapped cavity mechanism were captured prior to mobilising deep flow-round mechanism. In contrast to White et al. (2010), a trapped cavity mechanism was identified in between shallow and deep flow-round mechanism.
The critical depths of forming a trapped cavity mechanism and mobilising deep flow-round mechanism were quantified, and hence the depth span of shallow failure mechanism (stage I) and trapped cavity mechanism (stage II).

Both shallow and trapped cavity mechanisms provided lower T-bar resistance factors and hence higher undrained shear strength compared to the deep flow-round mechanism provided resistance factor.

T-bar roughness factor was found to increase the (i) critical depths of forming trapped cavity mechanism and depth span of the trapped cavity, and (ii) more reduction on resistance factor at trapped cavity mechanism stage compared to that of full-flow mechanism.

For interpreting the undrained shear strength profile from T-bar penetration resistance in single layer clay, a new framework was proposed, accounting for the shallow and trapped cavity mechanisms with different soil undrained shear strength level (Figure 5-14).

It is worth to mention that the adopted Tresca model does not consider strain softening and strain rate dependency. The combined effect of rate dependency and progressive softening has been investigated extensively in Einav & Randolph (2005) and Zhou & Randolph (2007, 2009). Following their work, Zhou et al. (2016) has conducted a parallel comparison using the basic Tresca model and Tresca model with implemented strain softening and rate dependency with typical kaolin clay parameters on ball penetrometers, which conclude a difference of less than 3% on resistance factor. Therefore, the proposed framework is considered suitable for low sensitivity clays. For high-sensitivity clays, further investigation is required to make corrections for effect of strain softening and rate dependency.
Effect of trapped cavity mechanism on T-bar penetration resistance in uniform clays

5.7 References


Chapter 5


Effect of trapped cavity mechanism on T-bar penetration resistance in uniform clays
Stewart, D. P. (1992). Lateral loading of piled bridge abutments due to embankment

*Journal of Geotechnical Engineering* **120**, No. 12, 2230-2235.


penetrometer tests at shallow embedment and in very soft soils. *Canadian

Zhou, H. & Randolph, M. F. (2009). Numerical investigations into cycling of full-

Table 5-1. Summary of LDFE analyses performed on uniform clays

<table>
<thead>
<tr>
<th>Group ID</th>
<th>Scale</th>
<th>$D$ (m)</th>
<th>$s_u$ (kPa)</th>
<th>$s_u/\gamma'D$</th>
<th>$\alpha$</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Field</td>
<td>0.04</td>
<td>10</td>
<td>41.6</td>
<td>0; 0.1; 0.2; 0.3; 0.4; 0.5; 0.6; 0.7; 0.8; 0.9; 1</td>
<td>Pre-embedded LDFE analysis; resistance factor validation of the LDFE model</td>
</tr>
<tr>
<td>II</td>
<td>Centrifuge</td>
<td>0.75</td>
<td>29.1</td>
<td>5.24</td>
<td>0.3</td>
<td>With the same layering condition in Chapter 2; mechanism validation of the LDFE model</td>
</tr>
<tr>
<td>III</td>
<td>Centrifuge</td>
<td>0.75</td>
<td>13.5</td>
<td>3</td>
<td>0.3</td>
<td>Evolvement of failure mechanism with T-bar penetration</td>
</tr>
<tr>
<td>IV</td>
<td>Centrifuge</td>
<td>0.75</td>
<td>3; 4.5; 9; 13.5; 18; 22.5; 27; 31.5</td>
<td>0.67; 1; 2; 3; 4; 5; 6; 7</td>
<td>0.3</td>
<td>Effect of shear strength on trapped cavity mechanism of centrifuge scale T-bar</td>
</tr>
<tr>
<td>V</td>
<td>Field</td>
<td>0.04</td>
<td>2; 4; 5; 10; 15; 20; 25</td>
<td>8.3; 16.7; 20.8; 41.6; 62.5; 83.3; 104.2</td>
<td>0.3</td>
<td>Effect of shear strength on trapped cavity mechanism of field scale T-bar</td>
</tr>
<tr>
<td>VI</td>
<td>Field</td>
<td>0.04</td>
<td>10</td>
<td>41.6</td>
<td>0; 0.1; 0.3; 0.4; 0.5; 0.7</td>
<td>Effect of roughness on the resistance factor of deep cavity mechanism</td>
</tr>
<tr>
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<td>Centrifuge</td>
<td>0.75</td>
<td>4.5; 9; 13.5; 18; 31.5</td>
<td>1; 2; 3; 5; 7</td>
<td>0.7</td>
<td>Effect of roughness on trapped cavity mechanism and resistance profile</td>
</tr>
<tr>
<td>VIII</td>
<td>Field</td>
<td>0.04</td>
<td>10; 15; 20; 25</td>
<td>41.6; 62.5; 83.3; 104.2</td>
<td>0.7</td>
<td>Effect of roughness on trapped cavity mechanism and resistance profile</td>
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</tbody>
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Table 5-2. Comparison of resistance factor of T-bar with and without cavity influence

<table>
<thead>
<tr>
<th>Roughness ($\alpha$)</th>
<th>0</th>
<th>0.1</th>
<th>0.3</th>
<th>0.4</th>
<th>0.5</th>
<th>0.7</th>
</tr>
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<tr>
<td>$h_t$ (for $s_u/\gamma'D = 41.6$; Group VI)</td>
<td>8.2</td>
<td>8.5</td>
<td>9.5</td>
<td>10.0</td>
<td>11.5</td>
<td>12.6</td>
</tr>
<tr>
<td>$N_{cf}$ (penetration from mudline, with cavity)</td>
<td>8.73</td>
<td>9.05</td>
<td>9.75</td>
<td>9.85</td>
<td>10.05</td>
<td>10.35</td>
</tr>
<tr>
<td>$N_{lw}$ (pre-embedded, without cavity)</td>
<td>9.31</td>
<td>9.79</td>
<td>10.61</td>
<td>10.97</td>
<td>11.12</td>
<td>11.88</td>
</tr>
<tr>
<td>Difference</td>
<td>6%</td>
<td>8%</td>
<td>8%</td>
<td>10%</td>
<td>10%</td>
<td>13%</td>
</tr>
</tbody>
</table>
Figure 5-1. Schematic figures of available geotechnical penetrometers for in-situ testing
Effect of trapped cavity mechanism on T-bar penetration resistance in uniform clays
Figure 5-2. Existing mechanisms for T-bar penetrometer: (a) full-flow mechanism (after Randolph & Houlsby, 1984); (b) pre-embedment less than half diameter (after Murff et al., 1989); (c) pre-embedment larger than half diameter (after Aubeny et al., 2005); (d) shallow failure mechanism and flow-round mechanism (after White et al., 2010); (e) trapped cavity mechanism (after Tho et al., 2011)
Figure 5-3. Comparison between plasticity bound solutions and pre-embedded LDFE results with varying roughness (Group I, Table 5-1)
Figure 5-4. Validation of soil failure mechanism against centrifuge observation
($s_u/\gamma'D = 5.24$; Group II, Table 5-1): (a) $d/D = 1.5$; (b) $d/D = 6.7$; (c) $d/D = 9.8$
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Figure 5-5. Development of a trapped cavity mechanism at (s_u/γ'D = 3; Group III, Table 5-1): (a) d/D = 0.5; (b) d/D = 1.5; (c) d/D = 4.4; (d) d/D = 5.9; (e) d/D = 10.1; (f) d/D = 10.5
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Figure 5-6. Depth required to mobilise trapped cavity mechanism and full flow mechanism (Groups IV and V, Table 5-1)
Effect of trapped cavity mechanism on T-bar penetration resistance in uniform clays

\[ s_u / \gamma D = 0.67 \]

- Red line: LDDE
- Dashed line: Equation 5-7
- Dotted line: White et al. (2010)

\[ N_{ff} = 10.61 \]

Stage I

Stage III

(a)
Figure 5-7. Typical resistance profiles with evolving mechanism: (a) scenario 1; (b) scenario 2 and 3
Effect of trapped cavity mechanism on T-bar penetration resistance in uniform clays

Figure 5-8. Evolving mechanisms and resistance factor stages of T-bar penetrometer
Figure 5-9. Fitting curve of T-bar resistance factor for shallow failure mechanism
Effect of trapped cavity mechanism on T-bar penetration resistance in uniform clays

Figure 5-10. Resistance factor profiles with varying roughness factor \( \frac{s_u}{\gamma'D} = 41.6; \) Group VI, Table 5-1)
Figure 5-11. Effect of roughness on duration of trapped cavity mechanism

(Group VII and VIII, Table 5-1)
Figure 5-12. Resistance factor profiles for T-bar with $\alpha = 0.7$ (Groups VII and VIII, Table 5-1)
Figure 5-13. Boundary of trapped cavity for varying roughness at $d/D = 16$ ($s_d/\gamma'D = 41.6$; Group VI, Table 5-1)
Effect of trapped cavity mechanism on T-bar penetration resistance in uniform clays

Figure 5-14. Recommended procedure for interpretation of undrained shear strength in single layer clay from T-bar resistance data
Figure 5-15. Example of interpretation using proposed framework for a centrifuge T-bar penetration data ($D = 1$ m)
CHAPTER 6  INTERPRETATION OF T-BAR PENETRATION DATA IN TWO-LAYER CLAYS

ABSTRACT

Accurate interpretation of the T-bar penetrometer data in layered soils is of great interests to the geotechnical community. This chapter studies the interpretation of undrained shear strength and layer boundaries from the T-bar penetration resistance profiles in both soft-stiff and stiff-soft clay sediments. Large deformation finite-element (LDFE) analyses were performed for the T-bar penetrating continuously from the soil surface. The proposed interpretation framework was based on extensive parametric LDFE analyses by varying soil strength ratio between the two layers, and the thickness of the top layer relative to the T-bar diameter. The evolution of the mobilised soil flow mechanisms with the progress of the T-bar penetration was illustrated, and the key features were compared with the visual observations (i.e. PIV analysis) from centrifuge tests. For soft-stiff clay deposits, squeezing mechanism dominated the behaviour when the T-bar approached the layer interface. There was no trapped soil plug from the top layer beneath the advancing T-bar penetrating in the bottom stiff layer. The interface can be identified as the kink point with sharp increase of the resistance profile sensing the bottom layer. For stiff-soft clay deposits, the enhanced downward soil movement, being attracted by the bottom soft layer, led to deformation of the layer boundary. A stiff soil plug was trapped at the base of the T-bar and penetrated into the bottom soft layer. The normalised resistance profiles dropped before reaching the layer interface sensing the bottom soft layer. A minimum
depth of $25D$ was needed to mobilise the ultimate resistance of the top layer. The layer boundary can be identified as $4.2D$ above the kink point on the resistance profile in the bottom layer. The stabilised resistance in the bottom layer increased with increasing strength ratio between the top and bottom layer due to a thicker stiff soil plug.

### 6.1 Introduction

In-situ testing is crucial for offshore site investigation due to the difficulty and expense involved in obtaining high-quality soil samples for laboratory testing. Since its first launch, the novel T-bar penetrometer has been attracting increasing attention for profiling the shear strength of fine-grained seabed sediments due to its superior features such as high resolution for soft clay sediments, negligible effect of overburden pressure, and insensitivity to soil rigidity index (Lu et al., 2004; Chung, 2005).

The current interpretation of T-bar data relies largely on upper bound and lower bound plasticity solutions (in terms of bearing capacity factor $= \frac{q_{net}}{s_u}$) for a deeply embedded infinitely long cylindrical bar, or for a pile undergoing lateral translation (Randolph & Houlsby, 1984; Martin & Randolph, 2006; Randolph & Andersen, 2006). The upper bound and lower bound values have been reported for different object-soil interface roughness (fully smooth to fully rough with $\alpha = 0$ to 1). For shallow embedment depths, the solutions from plasticity solutions and small strain FE analyses have been reported by Murff et al. (1989), Randolph & White (2008), and Aubeny et al. (2005).

Large deformation finite element (LDFE) analyses have been carried out for a deeply embedded plain strain bar by Zhou & Randolph (2007, 2009), and for a bar penetrating
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continuously from the soil surface by Barbosa-Cruz & Randolph (2005), White et al. (2010), Tho et al. (2011), and Chapter 5 of this thesis. There has been no trapped cavity mechanism present from the interpretation framework for the full penetration depths proposed by White et al. (2010), which was rectified by Tho et al. (2011) considering a trapped cavity mechanism, but for a narrow range of normalised shear strength and an ideally smooth T-bar. A much wider and typical range of normalised strength and T-bar roughness were considered in Chapter 5 of this thesis.

However, these solutions are limited for single layer clay. A range of fine-grained seabed stratifications are encountered in the Sunda Shelf, offshore Malaysia, offshore Africa, and even in the Gulf of Mexico (Menzies & Roper, 2008; Osborne et al., 2011; Menzies & Lopez, 2011). To rationally interpret the T-bar penetration data, i.e. to identify the layer boundaries and strength for each identified layer, from in-situ site investigation in layered fine-grained seabed sediments, a study on the behaviour of T-bar in layered sediments is needed.

The T-bar penetration in stratified clay deposits has been investigated through centrifuge model tests and LDFE analyses. The results from centrifuge tests have been discussed in Chapter 2, and those from LDFE analyses are reported in this chapter. The behaviour of the T-bar penetrometer in soft-stiff and stiff-soft clay deposits is investigated. The evolving soil flow mechanisms are compared with the centrifuge observations. An extensive parametric study, encompassing a practical range of soil strength ratio between the two layers and the thickness of the top layer relative to the T-bar diameter, leads to establish a framework for the interpretation of the T-bar penetrometer data.
6.2 Numerical modelling

The T-bar was simplified as an infinitely long or plane strain bar, which is a commonly adopted assumption (Lu et al., 2000; White et al., 2010). This study has considered a T-bar of diameter $D = 0.04$ m (typical for the T-bar used in the field) penetrating continuously from the soil surface into two-layer clay deposits varying the undrained shear strengths of the layers ($s_{u1}$, $s_{u2}$) and the thickness of the top layer ($h_1$). The problem is illustrated schematically in Figure 6-1a, where $d$ is the penetration depth of the T-bar invert, $t$ is the distance of the T-bar invert from the layer interface, $h_1$ is the thickness of the top layer soil, and $s_{u1}$ and $s_{u2}$ are the soil strengths of the top and bottom layer, respectively.

The numerical method and software package adopted in this chapter are identical to those considered in Chapter 5. Soil was considered as elastic-perfect plastic material obeying a Tresca failure criterion, with the yield stress solely determined by the undrained shear stress. The elastic behaviour was defined by Poisson’s ratio of 0.49 and Young’s modules $E = 500s_u$ throughout the analyses for all soil profiles. A uniform submerged unit weight $\gamma' = 6$ kN/m$^3$ was adopted over the soil depth, representing a typical average value encountered in the field. Two-layer strength profiles with uniform undrained shear strength of $s_u = 5$–50 kPa for the layers were considered, covering a range of clay strength commonly encountered in hydrocarbon active seabed sediments such as Sunda Shelf of south-east Asia, the Gulf of Mexico, offshore Africa and offshore South America (Castleberry & Prebaharan, 1985; Quirós & Little, 2003; Randolph, 2004; Borel et al., 2005; Menzies & Lopez, 2011; Osborne et al., 2011).

Table 6-1 summarises the parameters considered for this parametric study.
The interface between the T-bar and soil was modelled using elastoplastic nodal joint elements (Herrmann, 1978), and the soil strength at the soil-T-bar interface was modelled as $a\sigma_u$, where $\alpha$ is the roughness factor, as defined by Randolph & Houlsby (1984). $\alpha = 0$ represents smooth interface and $\alpha = 1$ represents rough interface. For this study, an intermediate roughness condition ($\alpha = 0.3$) was adopted for the majority of the simulations, as it is believed to be typical for the field T-bar and miniature centrifuge T-bar (Randolph et al., 1998; Randolph, 2004).

LDFE analysis were conducted using the FE package AFENA (Carter & Balaam, 1995), originally developed at the University of Sydney, coupled with remeshing and interpolation technique RITSS (remeshing and interpolation technique with small strain; Hu & Randolph, 1998a). $H$-adaptive mesh refinement cycles (Hu & Randolph, 1998b) were implemented to optimise the mesh by minimising discretisation errors for it concentrating in the zones with high strains. This method falls within what are known as arbitrary Lagrangian-Eulerian (ALE) finite element methods (Ghosh & Kikuchi, 1991), whereby a series of small strain analysis increments (using AFENA) are combined with fully automatic remeshing of the entire domain, followed by interpolation of all field values (such as stresses and material properties) on the Gauss points from the old mesh to the new mesh. The applications of the LDFE/RITSS technique has proven its robustness simulating various geotechnical boundary value problems (Lu et al., 2001; Randolph et al., 2008; Zhou & Randolph, 2009; Hossain & Randolph, 2010; Zhou et al., 2013; Ma et al., 2015; Wang et al., 2015).

Penetration process of a T-bar from the soil surface is simulated by specifying incremental displacements. The displacement increment size and number of steps of small strain analysis between each remeshing were chosen such that the cumulative penetration in each remeshing stage remained in the small strain range and less than
half the minimum element size. Six-node triangle elements with three internal Gauss points were adopted in all analyses. The minimum element size surrounding the surface of the T-bar was taken as $2\%D$, which was obtained through a mesh convergence study. Hinge and roller conditions were applied at the base and vertical side of the soil domain, respectively.

Due to the symmetry of the problem, only half of the T-bar and the soil domain were modelled. Figure 6-1b shows the initial mesh with a minimal pre-embedment depth of $0.00001 \text{ m} (0.00025D)$, which was necessary to avoid computational instability during initial penetration in the soil. Darker shade in a soil layer in all figures stands for stiff soil and lighter for soft soil. The size of the domain was set as $50D$ in both width and height to ensure that the domain boundaries were well outside the soil plastic zone.

6.3 Results and discussion

6.3.1 T-bar in soft-stiff clay

6.3.1.1 Soil flow mechanisms

Typical soil failure mechanisms in soft-stiff clay deposit are discussed linking to the corresponding penetration resistance profile to highlight the effect of mobilised mechanism on the penetration resistance. Figure 6-2 shows a penetration resistance profile ($q/s_{u1}$) labelling 6 representative depths (a~f), and Figure 6-3 shows the soil failure mechanisms corresponding to those depths ($h_1/D = 9, s_{u1}/s_{u2} = 0.2; s_{u1}/'D = 20.8, s_{u2}/'D = 104.2; \text{ Group I, Table 6-1}$). Two uniform resistance profiles with strength identical to top and bottom layer are added for reference. Note, all the three $q/s_{u1}$ profiles represent bearing capacity factors with depth in reference to the top layer strength (i.e. $s_{u1} = 5 \text{ kPa}; s_{u1}/'D = 20.8$).
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The resistance profile in the top layer largely follows the response in single layer clay with strength identical to the top layer ($s_u = 5$ kPa) (Figure 6-2), indicating no bottom stiff clay involved in the soil flow mechanism (Figure 6-3a~c). As the T-bar penetrates $6D$ into the top layer ($d/D = 6$, Figure 6-3a), it shows a localised flow around mechanism with the cavity walls nearly vertical. With further penetration into the top layer, continual soil backflow pushes the cavity wall to touch the centreline at $\sim 1D$ above the T-bar top surface, leading to formation of the trapped cavity ($d/D = 8$, Figure 6-3b). A fully localised flow-round mechanism is mobilised with a trapped cavity above the advancing T-bar. The formation of a trapped cavity is consistent with that obtained in single layer clay through LDFE analyses (Figure 5-5b~d; Chapter 5). As discussed in Chapter 5, for uniform soil with $s_u/\gamma'D > 8.3$, trapped cavity is found to form and last for a long period. In the current analysis, the normalised undrained shear strength is $s_u/\gamma'D = 20.8 \geq 8.3$, hence the formation of the trapped cavity is expected for $d/D > 9.7$. Meanwhile, at point ‘b’, the resistance factor is marginally higher than that of single layer clay. The early formation of trapped cavity mechanism and higher resistance factor may be caused by the strengthened backflow when downwards movement was restricted by the bottom stiffer layer.

The T-bar resistance starts to increase significantly at just $0.15D$ above the soil layer interface, i.e. at $d/D = 8.85$ (point ‘c’ on Figure 6-2), where the soil deformation is still completely restricted in the top layer, and the soil flow is directed laterally outward and then around the T-bar ($d/D = 8.85$, Figure 6-3c). This soil flow pattern is referred to squeezing mechanism, causing increased degree of soil backflow above the T-bar, same as the squeezing mechanism revealed in centrifuge test (Figure 2-7d, Chapter 2).
Once the T-bar penetrates into the bottom stiff clay layer ($s_{u2}/\gamma'D = 104.2$), the T-bar resistance increases rapidly and then gradually reaches the stabilised value. More specifically, the profiles increase sharply up to $\sim2D$ beneath the interface, as the proportion of soil deformation in the soft layer decreases and that in the stiff layer increases. The soil deformation is predominantly directed downward followed by upward around the T-bar, leading to the heave of the top of the bottom stiff layer (or the soft-stiff layer interface) adjacent to the T-bar ($dl/D = 10$, Figure 6-3d). With the T-bar penetrating more than $\sim2D$ in the bottom layer, the rate of increase penetration resistance reduces as the T-bar is fully separated from the top layer, and the soil flow is concentrated in the bottom layer (see Figure 6-3e, f). Finally it merges with the response of the single layer clay with the strength identical to the bottom layer ($s_u = 25$ kPa). The higher undrained shear strength of the bottom layer prolongs the trapped cavity up to deep penetration in the bottom layer.

6.3.1.2 Parametric study

In order to explore the effect of the relative thickness of the top layer ($h_1$) on the normalised penetration resistance ($q/s_{u1}$) profile, the results of various thickness ratios $h_1/D = 6, 9, 10$ and $12$ are plotted in Figure 6-4 for a constant strength ratio of $s_{u1}/s_{u2} = 0.2$ ($s_{u1}/\gamma'D = 20.8$, $s_{u2}/\gamma'D = 104.2$; Group II, Table 6-1). Two penetration resistance profiles for the T-bar penetrating in single layer uniform clay with the undrained shear strengths of the top and bottom layers are also included in Figure 6-4 for comparison. In the top soft clay layer, regardless of the top layer thickness ratio, $h_1/D$, the normalised resistance profiles ($q/s_{u1}$) increases with depth and largely follows the profile for the corresponding single layer clay ($s_u = 5$ kPa). As shown in Figure 6-4, consistent with Figure 6-2, all the profiles deviate slightly from the single layer profile about $\sim1.2D$ above the soft-stiff layer interface and increases sharply just above the
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The T-bar resistance profile attains to the limiting value for \( h_1/D \geq 9 \). The limiting bearing capacity factor of \( q/s_u \) = 9.75, that corresponds to the trapped cavity mechanism, is consistent with the findings in Chapter 5 (referencing to Equation 5-9). After the increase of resistance profile during the transition stage from top soft layer to bottom stiff layer, finally, all the resistance profiles merge together with the profile in single clay layer of the strength of the bottom layer (\( s_u = 25 \) kPa) after a further penetration of around 12\( D \) into bottom layer. This is because no soil plug (from the top layer) was trapped at the base of the T-bar and the effect of the top soft soil diminished.

In order to demonstrate the effect of the strength ratio, penetration resistance profiles are plotted in Figure 6-5 for \( s_u/s_u = 0.2, 0.25 \) and 0.33 with \( h_1/D = 10 \) (\( s_u/\gamma'D = 26.0, s_u/\gamma'D = 78.1~130.2; \) Group III, Table 6-1). The strength ratios are designed by varying the strength of the bottom layer, while maintaining the top layer strength at \( s_u = 6.25 \) kPa \( (s_u/\gamma'D = 26.0) \). It is found that, regardless of strength ratio, all the \( q/s_u \) profiles are identical in the top layer with all the kink points (i.e. the point of sharp increase/decrease) merge together (at ~0.15\( D \) above the interface), suggesting that the influence distance of the bottom stiff soil is insensitive to the strength (or strength ratio) of the bottom layer.

Based on the presented results for T-bar in soft-stiff clay, it can be concluded that: (a) the bottom stiff layer has minimal influence on the T-bar resistance profile in the top soft layer. Thus, the shear strength of the top layer can be interpreted following the interpretation framework developed for single layer uniform clay in Chapter 5; (b) the soft-stiff layer interface can be identified at just ~0.15\( D \) below the kink on the resistance profile in the top layer; (c) the strength of the bottom layer can be interpreted
using only the stabilised resistance in that layer, under that assumption that the bottom layer is thick enough to reach the stabilised resistance in the layer.

6.3.2 T-bar in stiff-soft clay

6.3.2.1 Soil flow mechanisms

A typical penetration resistance profile in stiff-soft clay deposit \( (h_1/D = 8, s_{u1}/s_{u2} = 4; s_{u1}/\gamma'D = 104.2, s_{u2}/\gamma'D = 26.0; \text{ Group IV, Table 6-1}) \) is shown in Figure 6-6, with 6 points marked at representative penetration depths (a~f). The corresponding soil flow patterns of these six depths are shown in Figure 6-7. In the top layer, the T-bar resistance profile firstly increases with penetration depth following the single layer resistance profile (Figure 6-6), with shallow failure mechanism dominating, and some backflow and upheaving movement observed \((d/D = 2.5, \text{ Figure 6-7a})\). At the maximum resistance point (point ‘b’ on Figure 6-6), the soil downward deformation extends to the bottom layer, and soil backflow becomes restricted \((d/D = 3.2, \text{ Figure 6-7b})\).

Then the resistance starts to decrease and deviate from the single layer profile at a distance of \(4.8D\) above the layer interface, indicating the influence from the bottom soft layer. Finally, it reaches a stabilised value in the bottom layer at \(~4.2D\) under the interface (Figure 6-6). In this process \((d/D = 5, \text{ Figure 6-7c})\), the soil deformation is directed predominantly downward to the bottom soft layer, with little or no upward movement, leading to (i) vertical cavity above the advancing T-bar, (ii) significant bending of the stiff-soft layer interface. With the progress of the T-bar penetration, the proportion of soil deformation in the bottom layer and deformation of the interface increase \((d/D = 7.5; \text{ Figure 6-7d})\). This results in a continual decrease in penetration resistance. For the T-bar penetration in the bottom soft layer, a layer of stiff clay is dragged down with associated deformation of the interface \((d/D = 9; \text{ Figure 6-7e})\). It
is worth noting that in the top stiff layer, due to bottom soft layer attraction, the formation of trapped cavity is delayed compared to that in uniform clays. This is because the formation of trapped cavity, i.e. two cavity walls approaching the centreline, needs to be driven by backflow. However, in this case, soil mainly flow downward rather than backward. Finally, the T-bar is detached from the top soft layer, but a soil plug was trapped at the base of the T-bar \((d/D = 12.5); \text{Figure } 6-7f\). Owing to the influence of the stiff soil plug remaining at the base of the advancing T-bar (Figure 6-7f), this stabilised value is about 5% higher than that of the single layer. A localised flow-round flow mechanism is mobilised around the T-bar, whilst a trapped cavity remains open above the T-bar.

The phenomena of soil movements with early attraction to the bottom soft clay and the trapping of the top stiff soil plug in the bottom soft clay are consistent with those observed from the centrifuge tests reported in Chapter 2. However, the volume of the trapped stronger material seems to be greater than that observed in centrifuge testing (Figure 2-11f in Chapter 2). This may be caused by the stronger backflow associated with a much smaller normalised strength of the top layer \((s_{u1}/\gamma'D = 104.2 \text{ in LDFE vs } s_{u1}/\gamma'D = 5.2 \text{ in centrifuge test})\). The higher strength ratio in the centrifuge test is due to the larger prototype diameter \((D = 0.75 \text{ m})\) of the T-bar, relative to the field scale T-bar of \(D = 0.04 \text{ m} \) considered in the LDFE analysis.

An additional LDFE analysis was therefore carried out on the centrifuge T-bar of \(D = 0.75 \text{ m}, \) with strength and layering information same as TB2 in Table 2-1 \((h_1/D = 12.3,\ s_{u1}/s_{u2} = 3.96;\ s_{u1}/\gamma'D = 5.2,\ s_{u2}/\gamma'D = 1.4;\ \text{Group V, Table 6-1})\). As illustrated in Figure 6-8, the soil flow patterns from the LDFE analysis are reasonably consistent with those observed from the centrifuge test, with enhanced downward movement where the T-
bar approaches the interface (Figure 6-8a) and full flow-round mechanism is mobilised in the bottom layer (Figure 6-8b). Negligible trapping of stiff clay at the base of the T-bar can be seen from both PIV and LDFE analyses. Thus, this comparison validates the LDFE model. Some difference can also be observed between PIV analysis and numerical analysis on the shape of the trapped cavity, the stiff-soft boundary bending and the backflow mobilisation zone. It might be due to the incompetency of Tresca model in accounting for elasto-plastic and softening behaviour of soil, as well as water-soil interactions as in physical testing.

By comparing Figure 6-7f ($s_{u2}/\gamma' D = 26.0$ for field T-bar) and Figure 6-8b ($s_{u2}/\gamma' D = 1.4$ for centrifuge T-bar), another difference can be identified as the trapped cavity above the T-bar exists even after penetrating in the bottom layer by 4.5$D$ in Figure 6-7f. However, the T-bar top has been backfilled in Figure 6-8b with 2.4$D$ penetration in the bottom layer. This is because that the backflow process relies largely on $s_u/\gamma' D$ with earlier backflow resulted by lower $s_u/\gamma' D$ (Hossain et al., 2005). Since this chapter focuses on the interpretation of field T-bar tests, the following parametric studies are conducted on a field scale T-bar.

### 6.3.2.2 Parametric study

**Effect of top layer thickness ($h_1/D$)**

To demonstrate the effect of the relative thickness of the top layer, normalised penetration resistance profiles ($q/s_{u2}$) are plotted in Figure 6-9 for a range of $h_1/D$ from 8 to 25, but with identical $s_{u1}/s_{u2} = 4$ ($s_{u1}/\gamma' D = 104.2$, $s_{u2}/\gamma' D = 26.0$; Group VI, Table 6-1). The resistance profiles for single layers of equivalent strength with top and bottom layer clays are also included in Figure 6-9 for comparison. In general, in the top layer, the profiles deviate from the single layer profile at a significantly larger
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distance (≥ 4.8D) from the interface compared to that in soft-stiff clays (~0.15D). A local peak is formed and then drops sharply being attracted by the bottom soft layer. The distance of the peak resistance in the top layer to the layer interface (or the influence distance of the bottom soft clay layer) increases from 4.8D to 5.3D as \( h_1/D \) increases from 8D to 10D. This distance reaches to a stabilised number of 7.5D for \( h_1/D \geq 15 \), although a gradual decrease can be observed from as high as 15D away from the interface for \( h_1/D = 25 \).

Among all the cases investigated, the profile only for \( h_1/D = 25 \) attains the stabilised response of the top layer. As such, for interpreting shear strength, the stabilised resistance and limiting bearing capacity factor can be used for \( h_1/D \geq 25 \). For \( h_1/D \leq 25 \), the resistance profile prior to the peak and shallow bearing capacity factors should be used for interpretation according to the framework proposed in Chapter 5.

In the bottom layer, the rapid reduction in resistance continues. Eventually, all the profiles attain a kink consistently at a depth of ~4.2D below the interface, indicating that the interface can be identified at ~4.2D above the kink in the bottom layer regardless of \( h_1/D \). The stabilised normalised resistance factor in the bottom layer is around 5% higher than the corresponding normalised resistance factor of single layer soft clay. To distinguish them, normalised resistance factor for bottom layer of two-layer clay is defined as \( N_{t,b} = q/s_u/2 \) (‘b’ stands for bottom) and corresponding normalised resistance factor in single layer soft clay defined as \( N_{t,u} = q/s_u \) (‘u’ stands for uniform), with the difference between them denoted as \( F_{in} = (N_{t,b} - N_{t,u})/N_{t,u} \), i.e.

\[
N_{t,b} = N_{t,u} \times (1 + F_{in})
\]  

\[
s_{u/2} = \frac{q}{N_{t,b}}
\]
where $N_{tu}$ can be calculated as the stabilised value for the single layer soil referring to Chapter 5.

As shown in Figure 6-9, regardless of the top layer thickness, the stabilised value in the bottom layer are constant, i.e. $F_{in}$ is independent of $h_1/D$. This is because of trapping similar size of the stiff soil plug at the base of the T-bar, as shown in Figure 6-10.

*Effect of strength ratio ($s_{u1}/s_{u2}$)*

In order to explore the effect of the strength ratio on the penetration resistance profile, the results of various strength ratios $s_{u1}/s_{u2} = 4~8$, with $s_{u2} = 6.25$ kPa, are plotted in Figure 6-11 ($s_{u1}/\gamma'D = 104.2~208.3$, $s_{u2}/\gamma'D = 26$; Group VII, Table 6-1). Based on the above study on varying top layer thickness, the relative thickness of the top layer was kept constant at $h_1/D = 25$ (i.e. $h_1 = 1$ m) to ensure mobilisation of the stabilised resistance in the top layer. As shown in Figure 6-11, in the top layer, although the top layer strengths are different, all the resistance profiles reach to a peak value at a penetration depth around $10D$ (as marked on the profiles by squares), which is consistent with the findings in Chapter 5. Sensing the soft bottom layer influence, the resistance profiles start to reduce sharply from a distance of $t = 7.5D$ to $9.4D$ above the layer interface when $s_{u1}/s_{u2}$ increase from 4 to 8 (as marked on the profile by circles).

In the bottom layer, regardless of the strength ratio, all the profiles attain a stabilised resistance at ~$4.2D$ below the interface. This means that the influence depth below the stiff-soft layer interface is consistently ~$4.2D$, regardless of the strength ratio (Figure 6-11) and the top layer thickness ratio (Figure 6-9). In other words, the stiff-soft layer interface can be identified at ~$4.2D$ above the kink in the bottom layer. All the
Interpretation of T-bar penetration data in two-layer clays

Stabilised resistances are higher compared to the resistance on the single layer clay with the strength of the bottom layer \( (s_u = 6.25 \text{ kPa}) \), and the difference from that the single layer, \( F_{in} \), increases as the strength ratio \( (s_{u1}/s_{u2}) \) increases. This is because, in contrast to the finding for the top layer thickness ratio (Figure 6-10), the size of the trapped soil plug at the base of the advancing T-bar increases with increasing the strength ratio \( s_{u1}/s_{u2} \), as shown in Figure 6-12.

To facilitate the interpretation of the strength of the bottom soft layer, the influence of the stiff soil plug, \( F_{in} \), has been calculated from all the analyses in Figure 6-11. The values show an exponential trend under the considered condition of \( s_{u1}/s_{u2} \leq 8 \) (Figure 6-13) and can be expressed as

\[
F_{in} = 0.02\% \times \left( \frac{s_{u1}}{s_{u2}} \right)^{3.9} \quad (6-3)
\]

so that \( s_{u2} \) can be calculated by substituting Equations 6-2 and 6-3 into Equation 6-1.

Based on the above parametric studies on the stiff-soft clays, it can be concluded that:
(a) in the top layer, the resistance profiles deviate from the corresponding single layer clay profile due to the influence of the bottom soft soil, and the influence depth above the interface increases with \( s_{u1}/s_{u2} \) and \( h_{1}/D \); (b) when the top layer thickness is \( h_{1}/D \geq 25 \), the stabilised resistance of the top layer can be reached; (c) when the top layer thickness is \( h_{1}/D < 25 \), the thickness is not sufficient to mobilise the limiting resistance, and as such, the resistance profile prior to dropping and shallow bearing factors should be used for interpreting the top layer strength; (d) the influence depth below the interface is constantly \(~4.2D\) regardless of \( s_{u1}/s_{u2} \) or \( h_{1}/D \), and therefore the stiff-soft layer interface can be identified as \(~4.2\) above the kink point on the resistance profile in the bottom layer; (e) in the bottom layer, the stabilised resistance in the layered soils
are higher than that of the corresponding single layer clay, with the difference independent of \( h_1/D \), but affected by \( s_u1/s_u2 \), and Equation 6-1~6-3 can be used to obtain the strength of the bottom layer.

Based on the above findings, the full procedure for interpreting the undrained shear strength profile of stiff-soft clay sediments is proposed and shown in Figure 6-14.

### 6.4 Concluding remarks

This chapter reports the results of continuous penetration of T-bar penetrometer from the seabed surface into two-layer clay sediments through LDFE/RITSS approach. The extensive parametric study investigates the effect of top layer thickness and the strength ratio of both stiff-soft and soft-stiff clays.

For soft-stiff clay deposits, in the top layer, squeezing mechanism takes place when the T-bar invert is within \( ~1.2D \) from the interface. All the T-bar resistance profiles consistently, and regardless of the strength ratio and the thickness ratio of the top layer, show a kink at \( ~0.15D \) above the interface followed by increase sharply with the influence of the bottom stiff layer. The interface can therefore be identified at \( 0.15D \) beneath the kink. In the bottom layer, no soil plug is trapped at the base of the T-bar, and hence the stabilised value merges with the stabilised response of the corresponding single layer profile. The interpretation framework proposed for single layer clay in Chapter 5 can therefore be used.

For stiff-soft clay deposits, a top layer thickness greater than \( 25D \) is required to reach the stabilised the top layer strength. The resistance profile merges with the single layer profiles before decreasing and sensing the bottom soft clay layer. In the bottom layer, regardless of the top layer thickness and strength ratio, the resistance profiles stabilise
at ~4.2D below the interface, which can be used to identify the interface. The trapped stronger material at the base of the T-bar results in higher resistance factor compared to the response in single layer clay. This increase has been quantified against various strength ratios. A framework for interpreting the undrained shear strength profile for stiff-soft clay sediments has been proposed.

### 6.5 References


Chapter 6


Interpretation of T-bar penetration data in two-layer clays


Chapter 6


Table 6-1. Summary of LDFE analyses performed in two-layer clays

<table>
<thead>
<tr>
<th>Group ID</th>
<th>$D$ (m)</th>
<th>$s_{u1}$ (kPa)</th>
<th>$s_{u1}/\gamma'D$</th>
<th>$s_{u2}$ (kPa)</th>
<th>$s_{u2}/\gamma'D$</th>
<th>$s_{u1}/s_{u2}$</th>
<th>$h_1/D$</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>0.04</td>
<td>5</td>
<td>20.8</td>
<td>25</td>
<td>104.2</td>
<td>0.2</td>
<td>9</td>
<td>To reveal the flow mechanism in soft-stiff soil</td>
</tr>
<tr>
<td>II</td>
<td>0.04</td>
<td>5</td>
<td>20.8</td>
<td>25</td>
<td>104.2</td>
<td>0.2</td>
<td>6; 9; 10; 12</td>
<td>Soft-stiff soil; effect of top layer height</td>
</tr>
<tr>
<td>III</td>
<td>0.04</td>
<td>6.25</td>
<td>26.0</td>
<td>18.75; 25; 31.25</td>
<td>78.1; 104.2; 130.2</td>
<td>0.33; 0.25; 0.2</td>
<td>10</td>
<td>Soft-stiff soil; effect of strength ratio</td>
</tr>
<tr>
<td>IV</td>
<td>0.04</td>
<td>25</td>
<td>104.2</td>
<td>6.25</td>
<td>26.0</td>
<td>4</td>
<td>8</td>
<td>To reveal the flow mechanism in stiff-soft soil</td>
</tr>
<tr>
<td>V</td>
<td>0.75</td>
<td>29.1</td>
<td>5.2</td>
<td>7.5</td>
<td>1.4</td>
<td>3.96</td>
<td>12.3</td>
<td>To compare with centrifuge test results in Chapter 2</td>
</tr>
<tr>
<td>VI</td>
<td>0.04</td>
<td>25</td>
<td>104.2</td>
<td>6.25</td>
<td>26.0</td>
<td>4</td>
<td>8; 10; 15; 20; 25</td>
<td>Stiff-soft soil; effect of top layer height</td>
</tr>
<tr>
<td>VII</td>
<td>0.04</td>
<td>25; 31.25; 37.5; 43.75; 50</td>
<td>104.2; 130.2; 156.3; 182.3; 208.3</td>
<td>6.25</td>
<td>26.0</td>
<td>4; 5; 6; 7; 8</td>
<td>25</td>
<td>Stiff-soft soil; effect of strength ratio</td>
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Interpretation of T-bar penetration data in double layer clays

Figure 6-1. T-bar penetration in two-layer clays: (a) schematic diagram; (b) initial mesh of LDFE analysis
Figure 6-2. Typical resistance profile in soft-stiff clay \((h_1/D = 9, s_{u1}/s_{u2} = 0.2;\)

Group I, Table 6-1)
Interpretation of T-bar penetration data in double layer clays

(a)  (b)  

(c)  (d)
Figure 6-3. Soil flow mechanisms for T-bar penetration in soft-stiff clay \((h_1/D = 9, s_{u1}/s_{u2} = 0.2; \text{Group I, Table 6-1})\): (a) \(d/D = 6\); (b) \(d/D = 8\); (c) \(d/D = 8.85\); (d) \(d/D = 10\); (e) \(d/D = 11\); (f) \(d/D = 18\)
Interpretation of T-bar penetration data in double layer clays

Figure 6-4. Effect of top layer thickness on normalised penetration resistance in soft-stiff clays (Group II, Table 6-1)
Figure 6-5. Effect of strength ratio on normalised penetration resistance in soft-stiff clays (Group III, Table 6-1)
Figure 6-6. Typical resistance profile in soft-stiff clay ($h_1/D = 8$, $s_{u1}/s_{u2} = 4$; Group IV, Table 6-1)
Interpretation of T-bar penetration data in double layer clays

Figure 6-7. Soil flow mechanisms for T-bar penetration in stiff-soft clay ($h_1/D = 8$, $s_{u1}/s_{u2} = 4$; Group IV, Table 6-1): (a) $d/D = 2.5$; (b) $d/D = 3.2$; (c) $d/D = 5$; (d) $d/D = 7.5$; (e) $d/D = 9$; (f) $d/D = 12.5$
Figure 6-8. Soil flow mechanisms of centrifuge scale T-bar penetration in stiff-soft clay ($h_1/D = 12.3$, $s_{u1}/s_{u2} = 3.96$; Group V, Table 6-1): (a) $d/D = 12.4$; (b) $d/D = 14.7$
Figure 6-9. Effect of top layer thickness on normalised penetration resistance profiles in stiff-soft clays (Group VI, Table 6-1)
Figure 6-10. Effect of top layer thickness on trapped stiff clay plug in stiff-soft clays at 6D below layer interface: (a) $h_1/D = 8$; (b) $h_1/D = 10$; (c) $h_1/D = 15$ (Group VI, Table 6-1)
Figure 6-11. Effect of strength ratio on normalised penetration resistance in stiff-soft clays (Group VII, Table 6-1)
Figure 6-12. Effect of strength ratio on trapped stiff clay in stiff-soft clay plug at 4.5D below layer interface: (a) $s_{u1}/s_{u2} = 4$; (b) $s_{u1}/s_{u2} = 6$; (c) $s_{u1}/s_{u2} = 8$ (Group VII, Table 6-1)
Figure 6-13. Effect of strength ratio on normalised penetration resistance in stiff-soft clays
Figure 6-14. Procedure for interpretation of undrained shear strength from measured T-bar penetration resistance profile in stiff-soft clay
CHAPTER 7  END EFFECT OF T-BAR PENETROMETER USING 3D FE ANALYSIS

ABSTRACT

Accurate determination of soil in situ shear strength using the T-bar penetrometer requires full understanding of the soil flow characteristics around the T-bar, including the three-dimensional soil flow near the ends of the T-bar. The conventional plane strain solution of the T-bar resistance factor ignores the shearing that takes place around the ends of the T-bar, thus may blemish the accuracy of the interpretation. A three-dimensional finite element model has been built to investigate the end effect of the T-bar on its flow mechanism and resistance factor. The effects of the T-bar aspect ratio and surface roughness on resistance factor are explored extensively. A new interpretation formula of the T-bar resistance factor is proposed as a function of the T-bar aspect ratio and roughness.
7.1 Introduction

The recent exploration of hydrocarbon fields in deep water has placed more reliance on in situ testing, such as vane shear testing and push-in penetrometer testing, due to the difficulty and high cost of obtaining high-quality soil sample for laboratory testing. The T-bar penetrometer was initially developed for characterizing soft fine-grained soils in centrifuge testing (Stewart & Randolph, 1991) and has been adopted in the field for site investigation since the 1990s (Stewart & Randolph, 1994). Compared to the conventional push-in penetrometer, i.e. cone penetrometer, the advantage of the T-bar penetrometer includes: (a) a 5~10 times larger projected area, leading to higher resolution of penetration resistance in soft sediments; (b) negligible effect of overburden pressure and soil rigidity index on penetration resistance due to the full-flow mechanism (Lu, 2004, Zhou & Randolph, 2009). The T-bar penetrometer comprises a cylinder bar attached perpendicularly to the end of a smooth shaft, where a load cell has been embedded, as shown in Figure 7-1. Along with increasingly being used in offshore in situ site investigation, T-bar is also frequently used for soil strength determination in laboratory and centrifuge tests (Hossain et al., 2005, Dingle et al., 2008). Therefore, an accurate interpretation procedure of the T-bar data is of great interest to the geotechnical community.

The current recommended interpretation method is mostly based on previous analytical solutions for a pipe or pile, i.e. for an infinite long cylindrical bar, which has been solved as a plane strain problem (Randolph & Houlsby, 1984, Martin & Randolph, 2006). The net resistance $q_{net}$ of a bar deeply embedded in clay was linked directly to the undrained shear strength ($s_u$) of clay through a bar resistance factor $N_t$ according to,
The values of $N_t$ was presented as a function of the roughness factor ($\alpha$). Later, numerous two-dimensional numerical analyses on a fully embedded bar penetration have resulted resistance factors similar to those from limit analyses for various roughness (Lu et al., 2000, Randolph & Andersen, 2006, Zhou & Randolph, 2009). Despite these rigorous solutions, the plane strain solutions ignore the potential effects of real three-dimensional shape of the T-bar, which refers to shearing that may take place at the ends of the T-bar, and different failure mechanisms along the bar at different distance from the end, termed as ‘end effect’. These changes of mechanism may lead to changes in resistance factor and therefore would blemish the accuracy of the interpreted shear strength if not considered (see Equation 7-1).

The standard field T-bar penetrometer comprises a 40 mm diameter ($D$) and 250 mm long ($L$) aluminous cylindrical bar (for a shorter version $L = 160$ mm (Chung & Randolph, 2004)). For laboratory tests, a miniature T-bar has been designed with $D = 5$ mm and $L = 20$ mm. Low & Randolph (2010) also reported a T-bar of $D = 8$ mm and $L = 42$ mm in laboratory tests. Although various sized T-bar has been adopted, there are only a few studies that has given attention to resistance factors of T-bar with different geometry. Yafrate et al. (2007) conducted a series of in situ test on T-bar with various aspect ratios (i.e. length-diameter ratio, $L/D$) from 1.27 to 9.45 (DeJong et al., 2010, DeGroot et al., 2010). It was reported that the resistance factor of deeply embedded T-bar reduces from 10.9 to 7.8 as the aspect ratio increases from 1.28 to 6.25, although for a larger aspect ratio of 9.45, the resistance factor increases slightly, which is possibly due to the bending effect for a slender T-bar. A similar result has been reported by Chung (2005). The resistance factor of a deeply embedded T-bar

\[ s_u = \frac{q_{net}}{N_t} \]  

(7-1)
with $L/D = 8$ is around 5.3% lower compared to that of a T-bar with $L/D = 4$. And for results of T-bar with $L/D = 10$, the tendency does not apply because of the bending effect. However, there is currently no systematic study on corrections of resistance factor with different T-bar geometry.

In this chapter, a three-dimensional (3D) small strain finite element analysis was built to investigate the end effect on resistance factor of the T-bar and the soil flow mechanisms around the ends of the T-bar. A series of systematic parametric analyses in terms of aspect ratio ($L/D$) and roughness ($\alpha$) were conducted to quantify the end effect. A correction method accounting for the end effect was proposed to adjust analytical plane strain solutions.

### 7.2 3D numerical modelling

To explore the end effect on the mechanisms and resistance mobilized during penetration of a T-bar, a 3D finite element (FE) model was built with the commercial software Abaqus/Standard 6.14 (Dassault, 2014). This study considered the T-bar as a cylinder, which was modelled as a rigid body because the stiffness of the T-bar can be considered as infinite large compared to soil material. The T-bar was placed in the middle of the soil domain to study the deeply penetrated T-bar. Small strain analysis was conducted because, in uniform clay, the resistance-displacement response from small strain analysis was found to be similar to that from large strain analysis with a deep pre-embedding ($>4.5D$) (Lu, 2004). The utilisation of small-strain analysis when dealing with T-bar penetration problem can also be found in Randolph & Andersen (2006).

Considering the symmetry of the problem, only a quarter of the T-bar and soil domain were modelled. As shown in Figure 7-2, the soil regime was set to $40r$ ($r$ is the T-bar radius).
End effect of T-bar penetrometer using 3D FE analysis

radius) high, 20r wide and 1.5L long, which is proven large enough to eliminate boundary effect. At the sides of the model, no horizontal movement was permitted, and the model base plane was fixed in all three coordinate directions. The mesh comprised 8-node linear hexahedral elements with full integration (termed C3D8 in Abaqus). The mesh size around the T-bar was set to 0.1r after initial test on minimum element size (described in the following section) considering both computational efficiency and accuracy. A refined mesh zone of 8r surrounding the T-bar was generated. Figure 7-2 shows the finite element mesh for the T-bar of L/D = 6.25, which consists of approximately 87,000 elements.

Soil was considered as elastic-perfect plastic material obeying a Tresca failure criterion, with the yield stress solely determined by the undrained shear stress. A uniform undrained shear strength profile was considered in this study, so the undrained shear stress $s_u = 10$ kPa was assigned to the whole soil domain, although the stress level does not affect the normalized results presented in this chapter. The elastic behavior was defined by Poisson’s ratio of 0.49 and Young’s modules $E = 1000s_u$ throughout the soil profile. A uniform submerged unit weight $\gamma' = 6$ kN/m$^3$ was adopted over the soil depth, representing a typical average value of soft clay.

The relative surface roughness at soil-structure (i.e. soil-T-bar) interface is an estimation of the mobilised shear strength at the interface. Further physical meaning and determination of relative surface roughness can be found in DeJong et al. (2002). In this study, the soil strength at the soil-structure interface was modelled as $\alpha s_u$, where $\alpha$ is the roughness factor and $s_u$ is the soil undrained shear strength, which is consistent with Randolph & Houlsby (1984). Both smooth ($\alpha = 0$) and frictional ($\alpha = 0.1\sim1$) interface between the T-bar and the adjacent soil were modelled. For frictional interfaces, a general contact algorithm was used, specifying a Mohr-Coulomb friction
law combined with a limiting shear stress ($\tau_{\text{max}}$). The Coulomb friction coefficient was set to a high value ($\mu_C = 50$) to allow the value of limiting shear stress ($\tau_{\text{max}}$) to govern failure, which was set to $\alpha_s u$. In weightless soil, the penetration of the T-bar would cause tension force between the top half the T-bar and soil, which could not represent the T-bar deep penetration condition. This would also underestimate the friction force because negative pressure leads to zero friction under the Mohr-Coulomb friction law. To overcome this problem, a surcharge pressure equals to $10\delta_u$ is applied on the top of the soil domain to ensure positive pressure at the top interface.

### 7.3 Results and discussion

This section firstly presents the validation work of the 3D FE model against the existing plasticity bound solutions. Then the end effect is investigated extensively through comparison between two cases, with and without considering the end effect. Following this, a series of parametric analyses was performed to investigate the effects of $L/D$ and $\alpha$ on the end effect. Based on these results, a correction method on resistance factor was proposed to be used in practice.

#### 7.3.1 3D model validation

To validate the 3D model, the length of the soil domain was chosen as the same length as the T-bar ($6.25D$), which enables it to represent an infinite long T-bar, as shown in Figure 7-3a. In this case, every cross-section of the domain along the T-bar is essentially in plane strain condition. The resistance factor of this model is therefore comparable to plane strain plastic solutions. All the parameters of this infinite long model are the same as the model in Figure 7-2. A series of small strain analyses were firstly carried out to assess appropriate mesh density with the minimum element size ($h_{\text{min}}$) around the T-bar varying from $0.4r$ to $0.08r$ for both smooth ($\alpha = 0$) and rough
End effect of T-bar penetrometer using 3D FE analysis

($\alpha = 1$) conditions. The resistance factors ($N_t$) were obtained through normalizing the mobilized ultimate penetration force ($F$) by the T-bar projected area and soil undrained shear strength ($s_u$), i.e. $N_t = F/(r\times0.5L)/s_u$, and presented in Figure 7-3b. The upper bound solution by Martin and Randolph (2006) was also included in Figure 7-3b for comparison. It can be seen that $h_{\text{min}} = 0.1r$ is sufficiently small enough to obtain results close to the bound solution, and as such, $h_{\text{min}} = 0.1r$ was adopted for subsequent analyses. It can also be found that a penetration depth of $d = 0.1r$ is sufficiently large to achieve the ultimate capacity. The comparison of $N_t$ values for various $\alpha$ from this model and plastic bound solutions is presented in Figure 7-3c. It is shown that excellent agreement has been reached between the results from this study and the upper bound solution (Martin & Randolph, 2006), with maximum error being under 1.8%. This close agreement confirmed the robustness and good mesh quality of the model.

7.3.2 T-bar end effect

The end effect was investigated in-depth through the T-bar penetration with 3D model shown in Figure 7-2. The resulted resistance-displacement profile was compared with the infinite long model shown in Figure 7-3a to quantify how the end effect can impact the corresponding resistance factor. A typical T-bar used in practice with $L/D = 6.25$ and $\alpha = 0.3$ was considered in the 3D model. Figure 7-4 compares the load-displacement response of the T-bar penetration with and without the end effect. The resistance factor yielded from the 3D modelling is 10.83 while 10.36 was obtained from the infinite long modelling. This means, without accounting for the end effect, the simplified plane strain condition underestimates the resistance factor by 4.6%. This suggests that the interpreted soil strength ($s_u$) can be overestimated without considering the end effect.
A series of soil flow mechanism is illustrated in the following to investigate the end effect, with the aid of both longitudinal view (XZ-plane) and front view (YZ-plane), as indicated in Figure 7-2. To better display the mobilized soil failure mechanisms, cross-sections at different distance from the end of the T-bar (denoted as $x$) are defined in Figure 7-5, with the direction towards the middle of the T-bar taken positive, and the direction away from the T-bar negative. The positive distance was normalized by $L$ and the negative distance by $r$.

The flow mechanism for the T-bar without (Figure 7-6) and with (Figure 7-7) the end effect can be used to explain the different resistance factors shown in Figure 7-4. For the infinite long T-bar, the contour and vector plots in Figure 7-6b clearly shows, as expected, a full flow mechanism at the T-bar mid-section. From the longitudinal view in Figure 7-6a, all the contour bands are parallel along the T-bar, implying that the flow pattern of each cross-section along the T-bar is uniform. The legend of contour plot in Figure 7-6 also applies to the contour plots in all other figures in this chapter. However, with the presence of the end effect (Figure 7-7a), the T-bar penetration not only mobilizes the soil along the T-bar, but also shears the soil just beside the T-bar end. While the total displacement contour at the mid-section ($x = 0.5L$) still shows the same as the infinite long T-bar, the displaced zone decreases gradually along the bar towards the T-bar end, resulting in a much smaller displaced area at the end section ($x = 0$) (Figure 7-7b). At the mid-section of the T-bar, due to symmetry, the soil has to flow within the section itself. In contrast, at the end section, the soil can move in both cross-section plane and longitudinal plane, restricting the extent of the flow contours to a smaller area.

To provide a picture of how much soil beside the T-bar end were disturbed, a series of displacement contour plots of the cross-sections at different distances from the T-bar
End effect of T-bar penetrometer using 3D FE analysis

end are displayed in Figure 7-8, with the projected T-bar position marked by dashed lines. At the end of the T-bar, the mobilized soil failure zone is mostly restricted in the soil around the projected T-bar, rather than the soil attached to the end of the T-bar. This phenomenon is mostly because the T-bar considered here is relatively smooth ($\alpha = 0.3$). This flow pattern means that, at least for the relative smooth T-bar, the resistance contributed by the end effect is resulted mostly from the shear band around the projected T-bar, not from the friction between the end of the T-bar and the adjacent soil. This might also explains why Randolph & Andersen (2006) predicted only less than 1% of the total resistance would be caused by the end faces. Their prediction might only include the friction force at the end faces, neglecting more intensive soil failure happens around the projected T-bar and away from the end of the T-bar. With shear energy dissipation, the displacement becomes less and less intensive with increasing distance from the end of the T-bar (Figure 7-8b~c). Beyond -0.3$r$ from the end, the mobilized zone gradually diminished within 0.1$r$.

Although 3D soil flow magnitude can be unveiled from Figure 7-7~Figure 7-8, soil flow direction is yet to be discovered. Therefore, the soil flow mechanisms at the end section of the T-bar are shown in Figure 7-9, with the soil flow vectors at the end section ($x = 0$) are observed and displayed from longitudinal view ($x = 0$), and front view ($x = 0^+$) and the back view ($x = 0^-$). The longitudinal and front views are indicated in Figure 7-2, while the back view is the opposite direction of the front view as defined in Figure 7-5. The front view, together with back view, shows that the soil on the plane mostly flows around the T-bar. Note, in Figure 7-9a and c, the vectors at the top and bottom, respectively, are apparently ‘missing’ because those vectors direct perpendicular into the plane of the paper. This is consistent with the plane observed from the longitudinal view (Figure 7-9b), where the soil on the top part tends to move
towards the T-bar, and that on the bottom part moves away from the T-bar. In summary, the three sub-figures together suggest that the flow patterns of the soil beside the end of the T-bar is a combination of two components. One component directs around the end of the T-bar on the cross-section plane, being dragged to flow around by the adjacent soil around the T-bar. The other component, with advancement of the T-bar penetration, pushes the bottom soil away while allowing the soil on the top to move towards the end of the T-bar.

7.3.3 Effect of aspect ratio and roughness

The effects of T-bar geometry and interface roughness were investigated by a series of analyses, exploring a wide range of aspect ratio \((L/D = 4, 5, 6.25, 8, 10)\) and roughness \((\alpha = 0, 0.3, 0.5, 0.7, 1.0)\). For all the analyses, the diameter \((D)\) was kept constant with the field size T-bar \((0.04 \text{ m})\). The length of the cylinder was varied to study the effect of aspect ratio, covering both field sized \((L/D = 6.25)\) and laboratory sized \((L/D = 4)\) T-bar. Although the diameter of the laboratory T-bar is much smaller \((0.005 \text{ m})\), corresponding sensitivity analyses confirmed that the scale effect does not affect the results when the computing domain is in proportion to the size of the T-bar.

The results of this parametric study are summarized numerically in Table 7-1 and graphically in Figure 7-10, with \(L/D = \infty\) representing the infinite long T-bar with no end effect. In Figure 7-10, the solid lines marked with squares represent the results for the T-bars with the end effect and the corresponding dashed line represent those for the T-bars without the end effect. The contribution of the end effect to the resistance factor is quantified as \(\beta = (N_{t,3D} - N_{t,\text{ideal}})/N_{t,\text{ideal}}\times100\%\) (Table 7-1), where, \(N_{t,3D}\) refers to the T-bar resistance factor with limited aspect ratio \((L/D = 4\sim10)\), and \(N_{t,\text{ideal}}\) is the resistance factor of the infinite long T-bar \((L/D = \infty)\). It is seen that, overall, the contribution of the end effect, \(\beta\), on T-bar resistance factor is significant. For any value
of $\alpha$, $\beta$ is the highest for $L/D = 4$ and decreases with increasing $L/D$. For instance, the highest resistance factor of the T-bar with end effect is 13.20 for $\alpha = 1$ and $L/D = 4$, which is about 9.7% higher (i.e. $\beta = 9.7\%$) than that of the infinite long T-bar, and the gap gradually reduces to $\beta = 3.8\%$ for $L/D = 10$. For any $L/D$, $\beta$ increases with increasing $\alpha$. This suggests rough T-bar is more prone to be affected by the end effect. The reason can be explained with total displacement distribution at the T-bar end section for the T-bar with same aspect ratio but varying roughness (Figure 7-11). As expected, the extent of deformation of the soil adjacent to the end of the T-bar intensifies with increasing roughness factor (Figure 7-11). For fully rough interface with $\alpha = 1$ (Figure 7-11c), the deformation zone within T-bar projected area is more intense than the soil flow around the T-bar extension line. This is because with a relatively rough interface, the shearing between the T-bar end and soil will be more profound compared to a relatively smooth interface. The additional high shearing energy in this area leads to higher resistance factor.

Overall, the computed bar resistance factors accounting for the end effect are in good agreement with measured data from field tests and laboratory tests. Assembling a worldwide, high-quality database of lightly over-consolidated clays, Low et al. (2010) reported an average resistance factor for the field T-bar ($L/D = 6.25$) of 11.87 (with the range being 9.81~14.66), while using average of triaxial compression, simple shear and triaxial extension strength in Equation 7-1, which lie between the computed values for $\alpha = 0.5$ and 0.7.

7.3.4 A new interpreting formula including end effect

To develop a unified design expression for the modified resistance factor accounting for the coupled effect of surface roughness and T-bar geometry, end effect contribution
(β) is expressed in Figure 7-12 as a function of T-bar roughness (α) and aspect ratio (L/D).

The numerical results plotted in β – L/D space (Figure 7-12a) suggests a power relationship between β – L/D, and positive correlation between β – α. As such, the function of the end effect contribution can be presumed to the form

\[ \beta = a \left( \frac{L}{D} \right)^{b - c} \]  

(7-2)

where \( a, b \) and \( c \) are curve fitting constants. The best-fitted constants are obtained using the least-squares regressions method, leading to \( a = 0.41, b = 0.25 \) and \( c = 1.27 \). The fitted curves are also included in Figure 7-12 for comparison, which shows excellent agreements with the FE results with coefficient of determination \( R^2 = 0.995 \).

For application, the resistance factor of 3D T-bar can be considered as made up of a base value, corresponding to the classical upper bound plasticity solution (Martin and Randolph, 2006) for the given interface friction ratio (α), modified by two factors represent the effects of roughness and aspect ratio. Thus, the resistance factor of 3D T-bar can be expressed as

\[ N_{t,3D} = N_{t,ideal} \left( 1 + 0.41 \left( \frac{L}{D} \right)^{0.25a-1.27} \right) \]  

(7-3)

From laboratory tests, Low and Randolph (2010) reported an average resistance factor of 11 for a T-bar with \( D = 8 \) mm and \( L = 42 \) mm (L/D = 5.3). The corresponding computed resistance factor from this fitting equation is 10.95 when assuming a relatively smooth interface of the laboratory T-bar (α = 0.3), which is very close to their test results. Although this fitting equation is developed based on the FE results for \( L/D = 4\sim10 \), it has been tested that it is also applicable to a lower or higher aspect ratio, although for a higher aspect ratio the bending effect needs to be considered. For
the infinite long T-bar \((L/D = \infty)\), \(N_{t_{\text{3D}}} \) becomes equal to \(N_{t_{\text{ideal}}} \). Besides, Equation 7-3 can also be used when dealing with other cylinder penetration problems such as pile or pipeline with deep embedment.

### 7.4 Conclusions

The end effect of the T-bar penetrometer on both resistance and mobilized soil failure mechanisms was investigated through 3D finite element analyses. The effect of aspect ratio and roughness have been explored extensively, with a robust expression proposed for the modified resistance factor to be used in practice. The following conclusions can be drawn from this study.

1. Due to the end effect, the size of the soil deformation zone reduced gradually from the middle cross-section of the T-bar to the end section. The spatial soil flow consisted of two components. One component moved around the T-bar extension line in the cross-section plane, dragged by the adjacent soil around the T-bar. The other component, with the advancement of the T-bar penetration, pushed the bottom soil away to accommodate the soil on the top moving towards the bottom.

2. The T-bar resistance factor considering the end effect was found to be higher than that without considering the effect, meaning the soil strength can be overestimated if the end effect is not considered. The parametric study showed that the contribution of the end effect reduces with higher aspect ratio and increases with higher T-bar surface roughness.

3. The proposed analytical expression was intended to provide a modified resistance factor \((N_{t_{\text{3D}}} )\) based on classical upper bound plasticity solution, but
accounting for the end effect with varying roughness and aspect ratio. The expression (Equation 7-3) is very convenient to apply in interpreting undrained shear strength and/or in quantifying the overestimation of strength without considering the end effect.

7.5 References


End effect of T-bar penetrometer using 3D FE analysis


Chapter 7


Table 7-1. End effect with varying aspect ratio and roughness

<table>
<thead>
<tr>
<th>Roughness, $\alpha$</th>
<th>Aspect ratio, $L/D$</th>
<th>$\infty$</th>
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<th>5</th>
<th>6.25</th>
<th>8</th>
<th>10</th>
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<td>0</td>
<td>$N_{t,3D}$</td>
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<tr>
<td></td>
<td>$\beta$ (%)</td>
<td>--</td>
<td>6.9</td>
<td>5.2</td>
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<td>2.8</td>
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<td>0.3</td>
<td>$N_{t,3D}$</td>
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<td>10.98</td>
<td>10.83</td>
<td>10.71</td>
<td>10.62</td>
</tr>
<tr>
<td></td>
<td>$\beta$ (%)</td>
<td>--</td>
<td>7.7</td>
<td>5.9</td>
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<td>3.4</td>
<td>2.5</td>
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<tr>
<td></td>
<td>$\beta$ (%)</td>
<td>--</td>
<td>8.2</td>
<td>6.5</td>
<td>5.0</td>
<td>3.8</td>
<td>2.9</td>
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<td>7.0</td>
<td>5.5</td>
<td>4.2</td>
<td>3.2</td>
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<tr>
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<td>$\beta$ (%)</td>
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<td>9.7</td>
<td>7.8</td>
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Figure 7-1. Schematic diagram of T-bar penetrometer

Figure 7-2. 3D modelling of T-bar penetration and mesh discretization
End effect of T-bar penetrometer using 3D FE analysis

(a) T-bar and Refined mesh zone

(b) Upper bound solution for $\sigma = 0$; $1$
(Martin and Randolph, 2006)

Smooth, $\sigma = 0$

Rough, $\sigma = 1$

Resistance factor, $N_t$

Displacement of T-bar, $d/r$
Figure 7-3. Validation against plasticity bound solution: (a) 3D infinite long T-bar model; (b) the mesh density effect on convergence with varying minimum element size ($h_{\text{min}}$); (c) comparison between plasticity bound solutions and 3D FE results with varying roughness
End effect of T-bar penetrometer using 3D FE analysis

Figure 7-4. Load-displacement response of T-bar with and without end effect

Figure 7-5. Definition of cross-section positions
Figure 7-6. Soil failure mechanisms of infinite long T-bar: (a) 3D view; (b) front view ($x = 0.5L$)
Figure 7-7. Total displacement contour of T-bar with $L/D = 6.25$: (a) 3D view; (b) left sub-figure: end section ($x = 0$); middle sub-figure: longitudinal section; right sub-figure: middle section ($x = 0.5L$)
Figure 7-8. Total displacement contour of cross-sections at different distances away from T-bar end: (a) $x = 0$; (b) $x = -0.1r$; (c) $x = -0.2r$; (d) $x = -0.3r$

Figure 7-9. Soil failure mechanism of T-bar at end-section plane ($x = 0$): (a) back view ($x = 0^-$); (b) longitudinal view ($x = 0$); (c) front view ($x = 0^+$)
Figure 7-10. Resistance factor of T-bar with varying aspect ratio and roughness

![Graph showing resistance factor vs. length-diameter ratio for different L/D values and roughness factors.](image)

Figure 7-11. Total displacement contour at T-bar end section with varying roughness ($x = 0$): (a) $\alpha = 0.3$; (b) $\alpha = 0.7$; (b) $\alpha = 1.0$
Figure 7-12. 3D FE results with fitted curves
CHAPTER 8 CONCLUDING REMARKS

8.1 Introduction

This thesis has described a series of studies on the behaviour of T-bar, ball and cone penetrometers in both uniform and layered sediments through experimental and numerical analysis. Firstly, for the three penetrometers, centrifuge tests of their penetration in layered clay sediments were conducted, allowing the associated soil failure mechanisms to be captured by a camera and quantified through particle image velocimetry (PIV) analyses. Secondly, for T-bar penetrometer, extensive parametric study of its penetration in both uniform single and layered clays was through large deformation finite element (LDFE) analyses. Thirdly, the end effect of the T-bar penetrometer was investigated through 3D-FE modelling. The main findings from this research are summarised below, followed by recommendations for future work.

8.2 Main findings

The main objectives of this study were (i) to achieve a better understanding of the behaviour of cone, T-bar and ball penetrometers in single and layered clays; and (ii) to establish interpretation frameworks for more accurate assessment of undrained shear strength in each individual layer and locations of layer boundary between adjacent layers through T-bar penetration resistance profiles. These objectives listed in Chapter 1 have been achieved successfully. The main findings related to each penetrometer are summarised below.
8.2.1 Main findings on T-bar penetrometer

8.2.1.1 Visualisation of soil flow mechanism of T-bar penetrometer in layered clay

– Centrifuge PIV analysis

The soil flow mechanisms have been studied by a series of centrifuges tests on the T-bar penetrometer in four types of layered deposits: soft-stiff, stiff-soft, soft-stiff-soft, and stiff-soft-stiff clays. The images of T-bar penetrating into layered clays were captured by a camera through a transparent window and quantified by PIV analyses. The trajectories of soil at varying distances from the interface were tracked to quantify soil layering effect.

- At the initial stage of a T-bar penetrating into soil from its surface, a shallow failure mechanism of soil is mobilised and an open cavity is formed above the T-bar. The depth of stable open cavity is related to normalised undrained shear strength $s_u/\gamma D$.

- When the T-bar penetrates deeper into a thick stiff layer, a trapped cavity is formed below the open cavity and above the penetrating T-bar. To our knowledge, this is the first observations of the trapped cavity under a similar stress level to the field and under water in centrifuge.

- With further penetration of the T-bar, a flow-around mechanism is achieved with the T-bar deeper penetration in the bottom soft layer, and the trapped cavity closes up. This is the mechanism used for the classical analytical solutions for a deeply embedded T-bar.

- When the T-bar approaches a soft-stiff interface, a squeezing mechanism is mobilised in the soft clay above the interface, where the soft soil squeezed sideways prior to mobilising the bottom stiff clay. The trajectories of soil close
to soft-stiff interface shows significant reduction in vertical movement compared to soil away from the interface.

- When the T-bar approaches a stiff-soft interface, the soil flow is attracted to bottom soft soil so the soil beneath the T-bar is moving downward predominantly, leading to obvious deformation (or bending) of the interface.

- For all T-bar tests in layered soils, once the T-bar fully detached from the upper layer soil, no trapping from the upper layer was observed in the bottom layer.

The trapped cavity phenomenon observed in centrifuge tests has been studied further parametrically using LDFE analysis. The findings are shown below.

### 8.2.1.2 Interpretation of T-bar penetration in uniform clay—quantification of trapped cavity effect

After validating the LDFE/RITSS (Remeshing and interpolation technique with small strain) method against the measured centrifuge test data in terms of flow mechanism, and existing theoretical solutions in terms of deep bearing capacity factor, a systematic parametric study was carried out. The soil flow mechanism and trapped cavity formation during the T-bar penetration in uniform clay have been investigated. The effect of the trapped cavity formation on the T-bar bearing capacity factor was quantified. The findings are listed below.

- There are three stages of the T-bar penetration into a uniform clay: stage I – shallow penetration with an open cavity; stage II – intermediate penetration with a trapped cavity; stage III – deep penetration with a flow-round mechanism.

- There are three scenarios of soil mechanism evolution based on the normalised soil undrained shear strength \( \frac{s_u}{y'D} \): scenario 1 – low normalised undrained shear strength level \( \frac{s_u}{y'D} \leq 1.0 \) where T-bar experiences stage I & III soil
flow mechanism with negligible duration of trapped cavity; scenario 2 – intermediate normalised undrained shear strength level \(1 < s_u/\gamma D \leq 8.3\) where T-bar experience all three stages of soil flow mechanisms with the formation of trapped cavity; scenario 3 – high normalised undrained shear strength level \((s_u/\gamma D > 8.3)\) where T-bar experiences stage I & II and a full flow-around mechanism is not formed up to 30\(D\) due to its associated weak backflow. For scenario 1 and 2, the stabilised value on the resistance profile corresponds to the full-flow mechanism solution. However, for scenario 3, the stabilised value on the resistance profile is lower than that of full-flow mechanism.

- The penetration depths to start and close the trapped cavity in stage II are functions of the normalised soil strength \(s_u/\gamma D\) (Equations 5-3 & 5-4). These depths increase with increasing T-bar roughness. Meanwhile, the trapped cavity lasts longer (i.e. the span of stage II is longer) for rougher T-bar.

- The occurrence of the trapped cavity reduces the T-bar resistance factor \((N_t)\) by 6% for a smooth T-bar \((\alpha = 0)\) and by 13% for a partially rough T-bar \((\alpha = 0.7)\) (see Table 5-2) relative to that with a full flow-around mechanism.

Based on the findings above, a new framework is proposed to interpret the undrained shear strength from T-bar penetration resistance in uniform clay, accounting for all three stages of T-bar penetration and T-bar roughness (Figure 5-14).

### 8.2.1.3 Interpretation of T-bar penetration resistance in two-layer clay

LDFE analyses on the T-bar penetration in both soft-stiff and stiff-soft clay profiles were conducted. The effects of top layer thickness and the strength ratio of the clay layers on T-bar penetration response are investigated. Major findings are listed below.

**T-bar in soft-stiff clays**
Concluding remarks

- When the top layer is thick as $h_1 \geq 9D$, the resistance profile in the top layer could reach the stabilised value corresponding to single layer profile (i.e. stage II in scenario 3); when the top layer is thin as $h_1 < 9D$, shallow mechanism (stage I) needs to be considered in T-bar resistance profile.

- When T-bar is approaching the soil layer interface, the soft soil is squeezed sideways with minimal interface deformation. Thus, squeezing mechanism is dominant.

- The T-bar resistance profile in the top layer is not affected by the soil strength ratio of the top-to-bottom layer.

- Once the T-bar passes the layer interface, the soil resistance profile increases rapidly first and then gradually merges with the stabilised resistance of the corresponding single layer profile of the bottom layer. Thus, the soil strength can be interpreted using the procedure proposed for single layer clay in Chapter 5.

- Regardless of the strength ratio and top layer thickness ratio, the transition of the T-bar resistance profile from the top layer to the bottom layer starts from $0.15D$ above the interface, which can be used to locate the layer boundary. It takes around $12D$ below the interface to reach the stabilised resistance of the bottom layer.

**T-bar in stiff-soft clays**

- The thickness of the top layer $h_1 > 25D$ is required to mobilise the stabilised resistance of the layer (i.e. stage II in scenario 3).

- For $h_1 < 25D$, resistance factor associated with shallow failure need to be considered in T-bar resistance profile.
• When the T-bar approaches the layer interface, the dominating soil downward movement under T-bar leads to the bending of the layer interface. A stiff clay plug is trapped at the base of the T-bar when penetrating into the bottom soft clay. The trapped stiff clay plug results in 5%~55% (depending on the strength ratio as 4~8) higher resistance factor compared to a clean T-bar. A formula (Equation 6-3) has been proposed to quantify this effect.

• Regardless of the strength ratio and top layer thickness ratio, the resistance profiles stabilised at ~4.2D after passing the interface, which can be used to identify the layer boundary. The transition of the T-bar resistance profiles from the top stiff layer to the bottom soft layer starts from ~4.8D to 15D above the interface depending on the strength ratio and top layer thickness ratio.

• A procedure for interpreting the undrained shear strength of the bottom soft layer is proposed accordingly (see Figure 6-14).

It should be noted that the framework build on the simple Tresca model does not considers the effect of strain softening and rate dependency. Previous studies (Einav & Randolph (2005) and Zhou & Randolph (2007, 2009), Zhou et al. (2016)) has show a less than 3% difference on low sensitive clays (e.g. kaolin) and as much as 40%~50% larger for soil sensitivity $S_t = 5$~10. Therefore, further investigation on this problem would help improve the proposed framework to account for high sensitive clays.

8.2.1.4 End effect of T-bar on penetration resistance

The end effect on both resistance and mobilized soil failure mechanisms were investigated through 3D FE analyses with varying T-bar aspect ratio ($L/D = 4$~10) and roughness ($\alpha = 0$~1.0). Main findings are listed below.
• The end effect influences the soil flow mechanism along the whole bar, though the major influencing zone of the 3D soil failure concentrates in $0.3D$ away from the T-bar and $0.5D$ into the T-bar from the end.

• The spatial soil flow beyond the T-bar end consisted of two components. One component moved around the T-bar extension line in the cross-section plane, dragged by the adjacent soil around the T-bar. The other component, with the advancement of the T-bar penetration, pushed the bottom soil away to accommodate the soil on the top moving towards the bottom.

• The end effect increases the T-bar resistance factor, relative to the infinite long T-bar in 2D plane strain conditions, by 2% for a smooth ($\alpha = 0$) and long ($L/D = 10$) T-bar to 9.7% for a rough ($\alpha = 1.0$) and short ($L/D = 4$) T-bar. An interpretation formula is proposed to quantify the end effect (Equation 7-3).

8.2.2 Main findings on ball and cone penetrometers
The ball and cone penetrometers in single and layered clays were studied by a series of centrifuge model tests. The evolution of soil flow mechanisms around the penetrometers were visualised by half-model centrifuge PIV tests. The calibration of cone penetrometer (full model) has been conducted through both centrifuge hydraulic test and axial load test.

8.2.2.1 Soil flow mechanism of ball penetrometer in layered clay
The main findings on flow mechanisms of ball penetrometer from centrifuge testing are listed below, and the similarities and differences with another full-flow penetrometer T-bar are illustrated.

• At shallow penetration depth, a shallow failure mechanism with open cavity above the ball penetrometer is observed.
• At deep penetration depth, a combined mechanism of vertical flow, cavity expansion type of flow and rotational flow is dominant. It is the first time this combined mechanism revealed during the ball penetration process in centrifuge tests.

• Similar to T-bar, when the ball approaches a soft-stiff interface, a squeezing mechanism in the top soft clay is mobilised. When the ball approaches a stiff-soft interface, the soil downward movement towards the bottom soft layer is dominant that induces soil layer interface bending.

• A stiff soil plug trapped underneath the ball can be observed when it penetrates into the bottom soft clay from the top stiff clay, which was not observed for T-bar.

8.2.2.2 Soil flow mechanism of cone penetrometer in layered clay

This study has reported results from a series of centrifuge model tests investigating the evolution of soil failure mechanisms during a cone penetrometer penetration in uniform, two layer and three layer clay deposits. The main findings are listed below.

For single uniform layer

• In deep penetration, the soil failure zone concentrates around or just above the shoulder.

• By comparing with the centrifuge PIV results, the SSPM (shallow strain path method) can provide reasonable prediction on maximum lateral and vertical displacements, however the upheave movement is overestimated.

For layered deposits

• Similar to T-bar and ball, when the cone approaches soft-stiff clay interface, squeezing mechanism is observed in the top soft clay. When the cone
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approaches stiff-soft clay interface, the downwards soil movement towards to the bottom soft clay is dominant to induce the layer interface bending.

- Due to the pointy shape of the cone, there is no top stiff layer soil plug trapped underneath the cone tip when it penetrates in the stiff-soft or soft-stiff clay.

8.2.2.3  Ambient pressure calibration of cone penetrometer in centrifuge

The total cone resistance measured by CPT is commonly used to interpret soil undrained shear strength. The total cone resistance includes two parts: the shear resistance and the overburden pressure. Although calibration methods for both parts are available, which are axial load calibration (for the shear resistance) and ambient pressure calibration (for the overburden pressure), very often only the check-up of axial load calibration is performed under the assumption of the same calibration factor for the ambient pressure. From the calibration practise in this study, some findings are shown below.

- For load cells with the same (or similar) conversion factors from axial load and ambient pressure calibrations, the axial load calibration is sufficient for CPT data interpretation.

- For some load cells, the conversion calibration factors from both calibration tests can be quite different or even opposite (i.e. +ve and –ve factors). In this case, both calibrations are needed to provide rational CPT data interpretations.

- The hydraulic test performed in centrifuge works well for ambient pressure calibration.
Chapter 8

8.3 Recommendation for future work

This thesis revealed the flow mechanisms of penetrometers in layered clays and investigated the interpretation of the T-bar penetrometer resistance profile in uniform and layered clay deposits. However, more studies are needed to extend the current study to consider 3D effect during continuous penetration, to cover larger range of soil profiles and to apply more sophisticated models. The recommendations are listed below.

8.3.1 End effect and shaft effect during shallow failure

The end effect in the 3D FE study here has revealed the combination of rotational soil flow around T-bar and inward soil flow towards T-bar centre under pre-embedment condition (see Chapter 7) for a deeply embedded T-bar. However, the 3D FE model was small strain analysis, which did not consider cavity formation and closure during shallow penetration. This effect can be examined with 3D LDFE modelling with continuous penetration of T-bar from soil surface to evaluate this effect. With 3D LDFE modelling, the effect of shaft-bar area ratio can be also be examined.

8.3.2 The penetration of T-bar in normally consolidated (NC) and over-consolidated (OC) soil

The sediments considered in this thesis include uniform and layered sediments with a constant strength in each individual layer. However, normally consolidated (NC) and over-consolidated (OC) soils are commonly encountered in the field. The penetration of the T-bar in single layer NC or OC clays has been investigated by Lu (2004), which revealed that the resistance factor can be less than that in uniform soil because the softer soils can be dragged under the T-bar. This influence can be increased with increasing the T-bar roughness factor. However, in the remeshing step of RITSS in Lu
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(2004), the closure of any point behind the T-bar was treated as the fully closure of the gap, which neglects the trapped cavity stage as revealed in the centrifuge test presented in this study. Therefore, the effect of the trapped cavity should be examined in NC and OC clays. Preliminary study (strength gradient, \( k = 1 \text{ kPa/m} \)) shows that in NC soil, the soil easily flows back at shallow depth, and hence no trapped cavity stage needs to be considered, while in OC soil (undrained shear strength at mudline, \( s_{um} = 5 \text{ kPa} \)) the trapped cavity stage can be observed. This suggests that more parametric study on the T-bar in NC and OC soils is needed to establish corresponding interpretation formulas/framework. At the same time, it is also important to conduct experimental test on the T-bar penetrometers in NC and OC soils to verify the corresponding behaviours.

8.3.3 Numerical modelling of T-bar in clay with advanced constitutive models

Advanced soil models are required to capture soil sophisticated soil behaviors, such as structure effect, strain softening etc. Lu (2004) has demonstrated that MIT-S1 gives an almost unity ratio of ball/T-bar penetration resistance, which is closer to field test data compared to 1.2 that resulted from the Tresca/von Mises model. Recently modified Cam clay (MCC) model has been used for exploring the response of penetrometers (Mahmoodzadeh et al., 2014; 2015). The effect of strain rate and strain softening have been explored in Zhou & Randolph (2007) and Zhou & Randolph (2009). However, most of these studies have been focusing on the pre-embedment study of the penetrometer, which could not be applied to the T-bar full profile interpretation. Therefore, it would be beneficial to conduct further comprehensive studies on continuous T-bar penetration.
8.4 References


