LARGE DEFORMATION FINITE ELEMENT ANALYSIS ON CONE PENETRATION TEST IN LAYERED SAND-CLAY SOILS

by

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BEng (Hons)

The thesis is presented for the degree of Doctor of Philosophy

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2020
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Qiang Xie

2 May 2020
ABSTRACT

The cone penetration test (CPT) is the most commonly conducted test in geotechnical site investigations. The continuous penetration resistance profiles can be used to interpret soil stratifications and soil properties within layers. Past studies have noted that during its penetration, the cone resistance could sense not only the soil layer where the cone is embedded, but also the soil layers ahead and behind the cone. The conventional interpretation methodologies for CPT data are mainly based on the results derived from a cone in a single soil layer. Direct application of the methods to layered soils can result in faulty interpretation if the adjacent layers’ effects are not considered. This is exacerbated when a thin layer is encountered by the CPT. This thesis focuses on CPTs in layered soils where both sand and clay layers are involved.

As the cone penetration is a typical large deformation problem, Large Deformation Finite Element (LDFE) analysis with the remeshing and interpolation technique with small strain (RITSS) method was deployed to simulate continuous penetration of the cone penetrometer. For sand, an extended Critical State Mohr-Coulomb (CSMC) model was deployed to capture its stress dependent behaviour under drained conditions. The CSMC model was calibrated for boundary problems in this project. A simple Tresca failure criterion was deployed for clay under undrained conditions. The LDFE/RITSS method was validated against existing centrifuge test data, before extensive parametric studies were carried out.

In the comprehensive parametric studies, the layered soil profiles included: (1) stiff clay over soft clay; (2) sand over clay; (3) a thin sand layer embedded in a uniform clay; and (4) a thin sand layer sandwiched between two different but uniform clay layers. For soil profiles (1) and (2), the studies were focusing on mini-cone penetration tests that are used in centrifuge test, where the prototype cone diameter varies with the centrifuge
acceleration levels. For soil profiles (3) and (4), a standard cone in field applications (i.e. cone diameter = 0.0357 m) was studied. Cone penetration resistance profiles, soil flow mechanisms, soil stress fields and soil failure patterns were analyzed based on the LDFE/RITSS results.

It is found that the cone resistance profile within an individual soil layer was affected by the layer thickness and the layer stiffness ratios to the adjacent layers. When a sand layer was involved in the soil profiles, the stress development in the sand layer was the determining factor for cone resistance profile in the sand layer. A soft clay layer ahead of the cone penetration had more influences on the cone resistance in sand than a soft clay layer behind the penetrating cone. Based on the parametric study in each soil profile, guidelines to interpret the soil strength of each individual layer are proposed for the mini-cones used in centrifuge tests and for the standard cone used in geotechnical site investigations.
TABLE OF CONTENTS

THESIS DECLARATION ............................................................................................................ i
ABSTRACT ................................................................................................................................... ii
TABLE OF CONTENTS ............................................................................................................. iv
NOTATIONS AND ABBREVIATION ...................................................................................... ix
LIST OF FIGURES ................................................................................................................... xiii
LIST OF TABLES ..................................................................................................................... xix
ACKNOWLEDGEMENTS ......................................................................................................... xx
AUTHORSHIP DECLARATION ........................................................................................... xxii

1 INTRODUCTION ................................................................................................................. 1-1
   1.1 BACKGROUND .............................................................................................................. 1-1
   1.1.1 Why does the cone penetration test (CPT) need to be investigated? ..................... 1-1
   1.1.2 Why does CPT in layered soils need to be studied? .............................................. 1-2
   1.2 RESEARCH OBJECTIVES ........................................................................................ 1-3
       1.2.1 Objective I ........................................................................................................... 1-3
       1.2.2 Objective II ....................................................................................................... 1-3
       1.2.3 Objective III ...................................................................................................... 1-4
   1.3 THESIS OUTLINE ..................................................................................................... 1-4
   1.4 REFERENCE ................................................................................................................. 1-5

2 LDFE ANALYSIS ON CONE PENETRATION TEST IN STIFF OVER SOFT CLAY FOR CENTRIFUGE TEST .................................................................................................. 2-1
   ABSTRACT ............................................................................................................................ 2-1
   KEYWORDS .......................................................................................................................... 2-2
   2.1 INTRODUCTION ......................................................................................................... 2-3
       2.1.1 Background .......................................................................................................... 2-3
       2.1.2 Previous work ..................................................................................................... 2-4
       2.1.3 Objective of present work ............................................................................... 2-6
   2.2 NUMERICAL ANALYSIS ........................................................................................... 2-6
       2.2.1 Geometry and parameters .................................................................................. 2-6
       2.2.2 LDFE analysis .................................................................................................... 2-7
       2.2.3 LDFE model validation ..................................................................................... 2-8
   2.3 RESULTS AND DISCUSSION .................................................................................... 2-9
2.3.1 Soil failure mechanisms ......................................................................................... 2-9
2.3.2 Interpretation of shear strength of the top-stiff layer ........................................... 2-10
2.4 INTERPRETATION FRAMEWORK............................................................................ 2-13
2.5 CONCLUSION.......................................................................................................... 2-13
2.6 REFERENCES.......................................................................................................... 2-14

3 INTERPRETATION OF CPT DATA IN SAND OVER UNIFORM CLAY IN A
GEOTECHNICAL CENTRIFUGE- LDFE ANALYSIS..................................... 3-1

ABSTRACT ..................................................................................................................... 3-1

KEYWORDS ................................................................................................................. 3-2

3.1 INTRODUCTION .................................................................................................... 3-3

3.1.1 Background........................................................................................................ 3-3
3.1.2 Previous works .................................................................................................. 3-4
3.1.3 Objectives ......................................................................................................... 3-6

3.2 METHODOLOGY .................................................................................................. 3-7

3.2.1 RITSS method .................................................................................................. 3-7
3.2.2 Soil models ...................................................................................................... 3-7

3.3 NUMERICAL MODELLING AND VERIFICATION .............................................. 3-10

3.3.1 Model setup .................................................................................................... 3-10
3.3.2 Validation of the LDFE/RITSS analysis ......................................................... 3-11

3.4 LDFE RESULTS AND DISCUSSION .................................................................. 3-11

3.4.1 Six cone penetration stages ............................................................................. 3-12
3.4.2 Parametric analysis ......................................................................................... 3-14
3.4.3 Peak resistance ($q_p$) and peak distance ($H_p$) in sand .................................. 3-16
3.4.4 Distance $H_k$ in clay ................................................................................... 3-17

3.5 INTERPRETATION FRAMEWORK .................................................................... 3-18

3.5.1 Interpretation of the undrained shear strength of clay ...................................... 3-18
3.5.2 Interpretation of the layer interface ............................................................... 3-19
3.5.3 Interpretation of the relative density of sand .................................................. 3-19
3.5.4 Evaluation of the interpretation formulas against existing methods .............. 3-19

3.6 CONCLUSION ..................................................................................................... 3-20

3.7 REFERENCE ........................................................................................................... 3-21
4 CONE PENETRATION TEST IN A THIN SAND LAYER EMBEDDED IN A UNIFORM CLAY - LDFE ANALYSIS ................................................................. 4-1

ABSTRACT ......................................................................................................................... 4-1

KEYWORDS .......................................................................................................................... 4-2

4.1 INTRODUCTION ......................................................................................................... 4-3

4.2 PROBLEM DEFINITION AND STUDY PLAN .......................................................... 4-5

4.3 NUMERICAL METHODOLOGY AND VERIFICATION ......................................... 4-6

4.3.1 Large deformation finite element analysis ............................................................. 4-6
4.3.2 Constitutive model ................................................................................................. 4-7
4.3.3 Mesh set-up ............................................................................................................ 4-8
4.3.4 Verification ............................................................................................................. 4-9

4.4 RESULTS BY PENETRATION STAGES ................................................................. 4-10

4.4.1 Flow mechanism .................................................................................................. 4-11
4.4.2 Stress development ............................................................................................... 4-12
4.4.3 Stage parameter development .............................................................................. 4-13

4.5 RESULTS OF PARAMETRIC STUDY ..................................................................... 4-14

4.5.1 Effect of top layer thickness, H_t/D ....................................................................... 4-14
4.5.2 Effect of sand layer thickness, H_s/D ..................................................................... 4-15
4.5.3 Effect of undrained shear strength of the clay, s_u ................................................. 4-16
4.5.4 Effect of relative density of the sand, I_D .............................................................. 4-16

4.6 INTERPRETATION OF CPT DATA ......................................................................... 4-17

4.6.1 Identification of layer interface location .............................................................. 4-17
4.6.2 Interpretation of undrained shear strength of clay ................................................ 4-18
4.6.3 Interpretation of relative density of sand.............................................................. 4-19
4.6.4 Assessment against the conventional interpretation method................................ 4-20

4.7 CONCLUSION ............................................................................................................ 4-20

4.8 REFERENCE ............................................................................................................... 4-22

5 CONE PENETRATION TEST IN A THIN SAND LAYER SANDWICHED BETWEEN DIFFERENT CLAY LAYERS- LDFE ANALYSIS .................................................. 5-1

ABSTRACT ......................................................................................................................... 5-1

KEYWORDS .......................................................................................................................... 5-2

5.1 INTRODUCTION ......................................................................................................... 5-3

5.2 METHODOLOGY ........................................................................................................ 5-6
5.2.1 RITSS method ................................................................. 5-6
5.2.2 Soil constitutive model .................................................. 5-6
5.3 NUMERICAL MODEL AND PARAMETRIC STUDY .......... 5-8
5.4 RESULTS AND DISCUSSION ........................................... 5-9
  5.4.1 Typical resistance profile ........................................... 5-9
  5.4.2 Flow mechanism ....................................................... 5-11
  5.4.3 Further discussion on the profile characteristics ....... 5-11
  5.4.4 Effect of the undrained shear strength of clay, \( s_u \) and \( s_{ub} \) .... 5-13
  5.4.5 Effect of sand layer thickness, \( H_s \) ................................ 5-16
  5.4.6 Effect of sand relative density, \( I_D \) ............................. 5-17
5.5 SUGGESTIONS FOR INTERPRETING CPT DATA .......... 5-19
  5.5.1 Interpretation of the clay layers ................................. 5-19
  5.5.2 Relative density of the sand layer ............................... 5-19
  5.5.3 Identification of the very dense thin sand layer ........... 5-20
5.6 CONCLUSION ................................................................. 5-21
5.7 REFERENCE ................................................................. 5-22

6 EFFECT OF LARGE DEFORMATION ANALYSIS FOR SITE INVESTIGATION TOOL - CPT IN LAYERED SOILS ................................................................. 6-1
ABSTRACT .................................................................................. 6-1
KEYWORDS ..................................................................................... 6-2
6.1 INTRODUCTION ................................................................. 6-3
6.2 PROBLEM SETUP ............................................................... 6-4
6.3 METHODOLOGY ................................................................. 6-5
6.4 RESULTS & DISCUSSION .................................................... 6-6
  6.4.1 Comparison between small strain and full LDFE Analysis 6-6
  6.4.2 Comparison between partial and full LDFE analyses .... 6-8
6.5 CONCLUSION ................................................................. 6-11
6.6 REFERENCES ................................................................. 6-12

7 CONCLUSION .......................................................................... 7-1
7.1 SUMMARY ................................................................. 7-1
7.2 CONTRIBUTION I ......................................................... 7-1
7.3 CONTRIBUTION II ....................................................... 7-3
7.4 CONTRIBUTION III........................................................................................................ 7-5
7.5 FUTURE WORK RECOMMENDATIONS ................................................................. 7-6
7.6 REFERENCE................................................................................................................. 7-8
### NOTATIONS AND ABBREVIATION

<table>
<thead>
<tr>
<th>Roman</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Scaling factor in Equation 3-4, 4-4 and 5-3</td>
</tr>
<tr>
<td>$A_1$ to $A_{10}$ and $B_1$ to $B_{10}$</td>
<td>Symbols used to define various stages along a cone resistance profile</td>
</tr>
<tr>
<td>$B_0$</td>
<td>Regression coefficient in Equation 3-8</td>
</tr>
<tr>
<td>$b_1$ to $b_3$</td>
<td>Power coefficients in Equation 3-8</td>
</tr>
<tr>
<td>$C_0$ to $C_3$</td>
<td>Regression coefficient in Equation 4-10, 4-11 and 5-6</td>
</tr>
<tr>
<td>D</td>
<td>Cone diameter</td>
</tr>
<tr>
<td>d</td>
<td>Cone penetration depth</td>
</tr>
<tr>
<td></td>
<td>Vertical distance from surface of soil sample to a point in Figure 4-8</td>
</tr>
<tr>
<td>$D_0$</td>
<td>Regression coefficient in Equation 3-12</td>
</tr>
<tr>
<td>$d_1$ to $d_3$</td>
<td>Power coefficients in Equation 3-12</td>
</tr>
<tr>
<td>$D_c$</td>
<td>Diameter of cone penetrometer in Figure 4-8</td>
</tr>
<tr>
<td>$d_{kb}$</td>
<td>Bottom kink distance</td>
</tr>
<tr>
<td>$d_{kt}$</td>
<td>Top kink distance</td>
</tr>
<tr>
<td>$d_p$</td>
<td>Peak distance</td>
</tr>
<tr>
<td>$d_{pre}$</td>
<td>Depth of the pre-embedded cone</td>
</tr>
<tr>
<td>$d_s$</td>
<td>Penetration depth below the clay-sand interface in Chapter 4</td>
</tr>
<tr>
<td></td>
<td>Pre-embedded distance to the stiff layer in Chapter 6</td>
</tr>
<tr>
<td>E</td>
<td>Young’s modulus</td>
</tr>
<tr>
<td>e</td>
<td>Void ratio</td>
</tr>
<tr>
<td>$E_0$</td>
<td>Reference stiffness</td>
</tr>
<tr>
<td>$E_c$</td>
<td>Young’s modulus of clay</td>
</tr>
<tr>
<td>$e_c$</td>
<td>Critical void ratio</td>
</tr>
<tr>
<td>$E_{cb}$</td>
<td>Young’s modulus of bottom clay</td>
</tr>
<tr>
<td>$E_{ct}$</td>
<td>Young’s modulus of top clay</td>
</tr>
<tr>
<td>$e_{max}$</td>
<td>Maximum void ratio</td>
</tr>
<tr>
<td>$e_{min}$</td>
<td>Minimum void ratio</td>
</tr>
<tr>
<td>$E_s$</td>
<td>Young’s modulus of sand</td>
</tr>
<tr>
<td>$e_{\Gamma}$</td>
<td>Virgin void ratio</td>
</tr>
<tr>
<td>f</td>
<td>Yield function</td>
</tr>
<tr>
<td>$f_s$</td>
<td>Sleeve friction</td>
</tr>
<tr>
<td>g</td>
<td>Gravitational acceleration</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
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<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>$G$</td>
<td>Shear modulus</td>
</tr>
<tr>
<td>$H$</td>
<td>Thickness of the top layer in two–layered soils</td>
</tr>
<tr>
<td>$h^*$</td>
<td>Cone penetration depth measured from the top of the soil sample in Figure 4-9</td>
</tr>
<tr>
<td>$H_c$</td>
<td>Critical distance</td>
</tr>
<tr>
<td>$H_k$</td>
<td>Kink distance in two–layered soils</td>
</tr>
<tr>
<td>$H_p$</td>
<td>Peak distance in two–layered soils</td>
</tr>
<tr>
<td>$H_s$</td>
<td>Thickness of middle sand layer</td>
</tr>
<tr>
<td>$H_t$</td>
<td>Total distance</td>
</tr>
<tr>
<td>$H_{tl}$</td>
<td>Thickness of the top clay layer in Chapter 4 and Chapter 5</td>
</tr>
<tr>
<td>$I_D$ and $D_R$</td>
<td>Sand relative density</td>
</tr>
<tr>
<td>$I_D'$</td>
<td>Estimated sand relative density</td>
</tr>
<tr>
<td>$I_r$</td>
<td>Rigidity index</td>
</tr>
<tr>
<td>$K$</td>
<td>Bulk modulus</td>
</tr>
<tr>
<td>$K_H$</td>
<td>Correction factor</td>
</tr>
<tr>
<td>$m$</td>
<td>Curve shape controlling factor in dilative angle relationship</td>
</tr>
<tr>
<td>$n$</td>
<td>Curve shape controlling factor in dilative angle relationship</td>
</tr>
<tr>
<td>$N_b$</td>
<td>Bearing capacity factor</td>
</tr>
<tr>
<td>$N_{bq}$</td>
<td>The proposed cone bearing factor to interpret the undrained shear strength of the top clay layer by the measured peak cone resistance</td>
</tr>
<tr>
<td>$p$</td>
<td>Mean stress</td>
</tr>
<tr>
<td>$p'$</td>
<td>Mean effective stress</td>
</tr>
<tr>
<td>$p_0'$</td>
<td>Initial mean effective stress at the middle point of a layer</td>
</tr>
<tr>
<td>$p_a$</td>
<td>Atmospheric pressure</td>
</tr>
<tr>
<td>$q$</td>
<td>Deviator stress</td>
</tr>
<tr>
<td>$q_b$</td>
<td>Constant cone resistance measured in a bottom uniform clay layer</td>
</tr>
<tr>
<td>$q_c$</td>
<td>Cone tip resistance</td>
</tr>
<tr>
<td>$q_{clayult}$</td>
<td>Ultimate cone resistance in clay</td>
</tr>
<tr>
<td>$q_{max}$</td>
<td>Maximum deviator stress</td>
</tr>
<tr>
<td>$q_{net}$ and $q_n$</td>
<td>Net cone resistance</td>
</tr>
<tr>
<td>$q_p$</td>
<td>Peak cone resistance</td>
</tr>
<tr>
<td>$q_t$</td>
<td>Total cone resistance</td>
</tr>
<tr>
<td>$r$</td>
<td>Horizontal distance from a point to the cone penetration axis</td>
</tr>
<tr>
<td>$R^2$</td>
<td>R-squared (statistic)</td>
</tr>
</tbody>
</table>
\( r_c \)  
Radius of cone penetrometer in Figure 4-9

\( s_u \)  
Undrained shear strength

\( s_{ub} \)  
Undrained shear strength of a bottom clay layer

\( s_{ut} \)  
Undrained shear strength of a top clay layer

\( t \)  
Thickness of the middle stiff clay layer in Chapter 2, appeared in a cited study (Ma et al., 2015)

\( u \)  
Pore water pressure

\( x \)  
Horizontal distance from a point to the cone penetration axis in Figure 4-8

\( z \)  
Vertical distance from surface of sample to a point in Figure 4-9

**Greek**

\( \alpha \)  
Cone roughness

Power coefficient on sand stiffness modulus in Equation 3-10

Power coefficient on normalized shear strength in Equation 4-10

Factor for averaging undrained shear strengths in Equation 5-6

\( \beta \)  
Power coefficient on clay stiffness modulus in Equation 3-10

Power coefficient on normalized top layer thickness in Equation 4-10

\( \gamma \)  
Unit weight

\( \gamma_b \)  
Unit weight of a bottom clay layer

\( \gamma_s' \)  
Effective unit weight of sand

\( \gamma_t \)  
Unit weight of a top clay layer

\( \lambda \)  
Slope of the critical state line

\( \nu \)  
Poisson’s ratio

\( \nu_c \)  
Poisson’s ratio for clay

\( \nu_s \)  
Poisson’s ratio for sand

\( \xi \)  
Power coefficient in the formula of critical state line

\( \sigma_1 \)  
Major principle stress

\( \sigma_3 \)  
Minor principle stress

\( \sigma_m' \)  
Vertical stress at the middle point of a layer

\( \sigma'_{pv} \)  
Vertical effective stress at the peak resistance location

\( \sigma_s \)  
Applied surcharge pressure in Figure 4-9

\( \sigma_v' \)  
Vertical effective stress

\( \sigma_{v0} \)  
Overburden pressure

\( \phi \)  
Friction angle

\( \phi_c \)  
Critical state friction angle
<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>AFENA</td>
<td>A Finite Element Numerical Algorithm</td>
</tr>
<tr>
<td>ALE</td>
<td>Arbitrary Lagrangian-Eulerian</td>
</tr>
<tr>
<td>ASTM</td>
<td>American Society for Testing and Materials</td>
</tr>
<tr>
<td>CC</td>
<td>Chamber Calibration</td>
</tr>
<tr>
<td>CEL</td>
<td>Coupled Eulerian–Lagrangian</td>
</tr>
<tr>
<td>CPT</td>
<td>Cone Penetration Test</td>
</tr>
<tr>
<td>CPTu</td>
<td>Piezocone penetration test</td>
</tr>
<tr>
<td>CPU</td>
<td>Central Processing Unit</td>
</tr>
<tr>
<td>C-S</td>
<td>Clay-Sand</td>
</tr>
<tr>
<td>CSL</td>
<td>Critical State Line</td>
</tr>
<tr>
<td>CSMC</td>
<td>Critical State Mohr-Coulomb</td>
</tr>
<tr>
<td>DIC</td>
<td>Digital Image Correction</td>
</tr>
<tr>
<td>FE</td>
<td>Finite Element</td>
</tr>
<tr>
<td>FL</td>
<td>Failure load</td>
</tr>
<tr>
<td>IRTP</td>
<td>International Reference Test Procedure</td>
</tr>
<tr>
<td>LDFE</td>
<td>Large Deformation Finite Element</td>
</tr>
<tr>
<td>MC</td>
<td>Mohr-Coulomb</td>
</tr>
<tr>
<td>NC</td>
<td>Normally consolidated</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>S-C</td>
<td>Sand-Clay</td>
</tr>
<tr>
<td>SF</td>
<td>Super Fine</td>
</tr>
<tr>
<td>SS</td>
<td>Small Strain</td>
</tr>
<tr>
<td>UWA</td>
<td>University of Western Australia</td>
</tr>
</tbody>
</table>

$\Psi$ State parameter

$\psi$ Dilation angle
LIST OF FIGURES

Figure 1-1 Schematic of CPT ................................................................. 1-7
Figure 1-2. Soil strata in this study ......................................................... 1-7
Figure 1-3. Thesis structure and publication summary ......................... 1-8
Figure 2-1. Schematic diagram of cone penetration in stiff over soft clays ....... 2-16
Figure 2-2. Mesh setup of cone in two-layer clay .................................... 2-16
Figure 2-3. Validation against centrifuge test ........................................ 2-17
Figure 2-4. Resistance profile with marked stages \((H/D = 4, s_{ut}/s_{ub} = 4, \text{ base case in Table 2-1})\) ................................................................. 2-17
Figure 2-5. Soil flow mechanisms at different penetration stages ............ 2-18
Figure 2-6. Resistance profiles for (a) various top layer thickness ratio \((H/D)\) at \(s_{ut}/s_{ub} = 4\); (b) various soil layer strength ratio \((s_{ut}/s_{ub})\) at \(H/D = 4\) ................................................................. 2-19
Figure 2-7. Resistance profile with defined variables for interpretation purpose .. 2-19
Figure 2-8. Relationship between critical depth \(H_c/D\) and \(q_{p}/q_{b}\) ............... 2-20
Figure 2-9. Relationship between \(H_c/D\) and \(q_{p}/q_{b}\) for \(H/D \geq H_c/D\) ............... 2-20
Figure 2-10. Interpretation framework of \(s_{ut}\) and \(H\) ........................................ 2-21
Figure 3-1. Schematic diagram of the CPT in sand over clay soils in the centrifuge test ....... 3-25
Figure 3-2. Dilation angle variation with the state parameter .................. 3-25
Figure 3-3. Young’s modulus variation with the mean effective stress for \(I_0 = 30\%, 50\%\) and \(90\%\) ................................................................. 3-26
Figure 3-4. Mesh setup with the illustrated refined zone .......................... 3-26
Figure 3-5. Validation of the LDFE/RITSS analysis against the centrifuge data (Teh et al. 2010, in Table 3-2) ................................................................. 3-27
Figure 3-6. A typical cone resistance profile in sand over uniform clay (D = 1 m, H/D = 5, \( I_D = 70\% \) and \( s_u = 10 \text{kPa} \)) ................................................................. 3-27

Figure 3-7. Soil flow mechanisms at six stages from Stage A\(_1\) to Stage A\(_6\) (D = 1 m, H/D = 5, \( I_D = 70\% \) and \( s_u = 10 \text{kPa} \)) .................................................................................. 3-28

Figure 3-8. Mean stress contours for six stages from Stage A\(_1\) to Stage A\(_6\) (D = 1 m, H/D = 5, \( I_D = 70\% \) and \( s_u = 10 \text{kPa} \)) ........................................................................... 3-29

Figure 3-9. Deviator stress contours for six stages from Stage A\(_1\) to Stage A\(_6\) (D = 1 m, H/D = 5, \( I_D = 70\% \) and \( s_u = 10 \text{kPa} \)) ............................................................................. 3-30

Figure 3-10. Effect of the thickness of the top sand layer on the cone resistance profiles (Group I in Table 3-3, \( I_D = 70\% \), \( s_u = 10 \text{kPa} \), and D = 1 m) ................................................................. 3-30

Figure 3-11. Effect of the initial sand relative density on the cone penetration resistance profile (Group II in Table 3-3, H/D = 5, \( s_u = 10 \text{kPa} \), and D = 1 m) ................................................................. 3-31

Figure 3-12. Effect of the undrained shear strength of the clay on the cone penetration resistance profile (Group III in Table 3-3, H/D = 5, \( I_D = 50\% \), and D = 1 m) ................................................................. 3-31

Figure 3-13. Effect of the cone diameter on the resistance profiles (Group IV in Table 3-3, H/D = 5, \( I_D = 70\% \) and \( s_u = 5 \text{kPa} \)) .................................................................................. 3-32

Figure 3-14. Effect of the cone diameter on the soil displacement and state parameter contours at d/D = 2.5 with (a) D = 0.2 m and (b) D = 2.0 m (Group IV in Table 3-3, H/D = 5, \( I_D = 70\% \) and \( s_u = 5 \text{kPa} \)) .................................................................................. 3-33

Figure 3-15. (a) Coefficient \( b_2 \) in Equation 3-8; (b) Performance of Equation 3-8 ............... 3-34

Figure 3-16. (a) Relationship between \( H_p/D \) and H/D; (b) Function between \( H_p/D \) and H/D ... 3-34

Figure 3-17. Relationships between \( H_k/D \) and \( I_D \) and \( s_u \) (D = 1 m) (a) \( s_u = 5 \text{kPa} \); (b) \( s_u = 20 \text{kPa} \) ..................................................................................................................... 3-35

Figure 3-18. Expression of \( H_k/D \) as a function of the stiffness ratio (\( E_s^a/E_c^b \)) .................. 3-35
Figure 3-19. Interpretation results from the proposed formula (Equation 3-12) and the conventional formula (Equation 3-13)................................................................................................................................. 3-36

Figure 4-1. Schematic plot of the cone penetration in clay-sand-clay soil ......................... 4-24

Figure 4-2. (a) Critical state line; (b) Relationship between dilation and state parameters ...... 4-24

Figure 4-3. Mesh set-up of the cone penetrating into the bottom clay (H_t/D = 20, H_s/D = 5). 4-25

Figure 4-4. Verification of the numerical analysis against centrifuge test data from Lu (2008) .................................................................................................................................................... 4-25

Figure 4-5. Verification of the numerical analysis against centrifuge test data from Roy et al. (2019) ................................................................................................................................................ 4-26

Figure 4-6. A typical CPT resistance profile with 10 penetration stages (H_t/D = 20, H_s/D = 5, I_D = 60 % and s_u = 20 kPa)................................................................................................................................. 4-27

Figure 4-7. Soil flow mechanisms from Stage A_1 to A_10 (H_t/D = 20, H_s/D = 5, I_D = 60 % and s_u = 20 kPa) ........................................................................................................................................ 4-29

Figure 4-8. Soil flow mechanisms of cone penetration in soft-stiff-soft clay by PIV method: (a) d/D = 6.5; (b) d/D = 8.3; (c) d/D = 14.3; (d) d/D = 17.1, from Wang et al. (2020) (in the figures, d- vertical distance from surface of sample to a point; D_c- radius of cone penetrometer; x- horizontal distance from a point to the cone penetration axis;)......................... 4-30

Figure 4-9. Soil flow mechanisms of cone penetration in sand by DIC method: (a) d/D = 2; (b) d/D = 6; (c) d/D = 22, from Arshad et al., (2017) (in the figures, σ_s- applied surcharge pressure; D_R- relative density of the sand; h^* - cone penetration depth measured from the top of the soil sample; r- horizontal distance from a point to the cone penetration axis; r_c- radius of cone penetrometer; z- vertical distance from surface of sample to a point.)......................... 4-32

Figure 4-10. Contours of mean stress from Stage A_2 to A_10 (H_t/D = 20, H_s/D = 5, I_D = 60 % and s_u = 20 kPa)....................................................................................................................................................... 4-33
Figure 4-11. Contours of deviator stress from Stage A4 to A9 (Ht/D = 20, Hs/D = 5, ID = 60 % and su = 20 kPa).......................................................................................................................... 4-34

Figure 4-12. Contours of state parameter from Stage A4 to A9 (Ht/D = 20, Hs/D = 5, ID = 60 % and su = 20 kPa) .......................................................................................................................... 4-35

Figure 4-13. Effect of top layer thickness on cone resistance profile (su = 20 kPa, ID = 60%) (a) with normalized penetration depth of d/D; (b) with normalized penetration depth in sand layer of ds/D .................................................................................................................................. 4-36

Figure 4-14. Effect of top layer thickness on kink distances (a) kink in the top clay, dk(t) (b) kink in the bottom clay, dk(b) .................................................................................................................................. 4-36

Figure 4-15. Effect of sand layer thickness on cone resistance profile (Ht/D = 20, ID = 60 % and su = 20 kPa): (a) cone resistance profile, (b) normalized cone resistance profile ................. 4-37

Figure 4-16. Effect of undrained shear strength on cone resistance profile (ID = 60 %, Ht/D = 20 and Hs/D = 5).................................................................................................................................. 4-38

Figure 4-17. Effect of relative density of sand on cone resistance profile (su = 20 kPa, Ht/D = 20 and Hs/D = 5).................................................................................................................................. 4-39

Figure 4-18. (a) Interpretation formula for the relative density of sand; (b) Coefficient, β....... 4-39

Figure 4-19. Comparison between the interpretation results by the proposed formula in this paper (Equation 4-10) and the method by Youd and Idriss (2001)................................................. 4-40

Figure 5-1. Schematic graph of the CPT in a clay-sand-clay soil................................................. 5-24

Figure 5-2. Mesh setup for the CPT in clay-sand-clay ................................................................. 5-24

Figure 5-3. CPT resistance profile in clay-sand-clay soils (Ht/D = 20, Hs/D = 10 and ID = 60% in Table 5-2): (a) Case A: sut/sub = 8 and sut = 80 kPa; and (b) Case B: sut/sub = 0.125 and sut = 10 kPa (C-S refers to the clay-sand interface, while S-C refers to the sand-clay interface). 5-25
Figure 5-4. Soil flow mechanism from Stages A₁ to A₁₀ for Central Case A (Hₛ/D = 20, Hₛ/D = 10, sₜₓ/s₀ = 8, sₜₓ = 80 kPa and Iₓ = 60%, Table 5-2) ............................................................. 5-27

Figure 5-5. Soil flow mechanism from Stages B₁ to B₁₀ for Central Case B (Hₛ/D = 20, Hₛ/D = 10, sₜₓ/s₀ = 0.125 and Iₓ = 60% in Table 5-2) ............................................................. 5-29

Figure 5-6. Mean stress contours at (a) Stage A₁, (b) Stage A₂, (c) Stage B₁, and (d) Stage B₂ (MS refers to the mean stress) ............................................................. 5-30

Figure 5-7. Effect of undrained shear strength sₜₓ and s₀ on the cone resistance profile (Hₛ/D = 5, Iₓ = 60%, Group I and Group IV in Table 5-2), (a) Case A: sₜₓ = 80 kPa and s₀ = 10, 20, 40 and 60 kPa and (b) Case B: sₜₓ = 10 kPa and s₀ = 20, 40, 60 and 80 kPa ........................................................................ 5-31

Figure 5-8. Effect of sand layer thickness Hₛ/D on the cone resistance profile (Iₓ = 60%, Group I and Group IV in Table 5-2), (a) Case A: sₜₓ = 80 kPa and s₀ = 10 kPa and (b) Case B: sₜₓ = 10 kPa and s₀ = 80 kPa ................................................................................................. 5-32

Figure 5-9. Effect of sand relative density Iₓ on the cone resistance profile (Hₛ/D = 5, Group I and Group IV in Table 5-2; data from another study (Xie et al., 2020a)), (a) Case A: sₜₓ = 80 kPa and s₀ = 10 kPa (b) Case B: sₜₓ = 10 kPa and s₀ = 80 kPa and (c) Case U: Case B: sₜₓ = 20 kPa and s₀ = 20 kPa .......................................................................................................... 5-33

Figure 5-10. Stiffness development in the sand layer for Case A (sₜₓ = 80 kPa and s₀ = 10 kPa) with Iₓ = 60% and 90% ........................................................................................................... 5-35

Figure 5-11. Vertical normalized displacement comparison between Iₓ = 90% and 60% for sₜₓ = 80 kPa and s₀ = 10 kPa ........................................................................................................... 5-36

Figure 5-12. Relationship between the peak resistance in sand and the factored undrained strength of the surrounding clay (α = 0.3) .................................................................................. 5-37

Figure 6-1. Schematic plot of CPT in soft-stiff-soft soil ........................................................................ 6-13
Figure 6-2. Load-displacement results of small strain analysis of CPT in soft-stiff-soft clay:
(a) net resistance, $q_c$, (b) resistance ratio, $q_c/s_u$. (FL: failure load)................................. 6-13

Figure 6-3. Comparison of cone resistance profiles by small strain and LDFE analyses for
CPT in soft-stiff-soft clays (C-C: clay-clay interface; SS: small strain analysis).............. 6-14

Figure 6-4. Load-displacement results of small strain FE analysis in clay-sand-clay soil ........ 6-14

Figure 6-5. Comparison of cone resistance profiles by small strain and LDFE analyses in
clay-sand-clay soils (C-S: clay-sand interface; S-C: sand-clay interface) ......................... 6-15

Figure 6-6. Comparison between the results of partial and full LDFE analysis for CPT in soft-
stiff-soft clays....................................................................................................................... 6-16

Figure 6-7. Mean stress development for partial ($d_{pre}/D = 15$) and full ($d_{pre}/D = 0$) LDFE
analysis of CPT in soft-stiff-soft clay .................................................................................. 6-18

Figure 6-8. Comparison between the results of partial and full LDFE analysis for CPT in clay-
sand-clay soils ....................................................................................................................... 6-19

Figure 6-9. Mean effective stress development for partial ($d_{pre}/D = 18$) and full ($d_{pre}/D = 0$)
LDFE analysis of CPT in clay-sand-clay soils................................................................. 6-20

Figure 7-1. Summary of contributions and their corresponding soil strata and cone .......... 7-9

Figure 7-2. Typical cone resistance profile of (a) two-layer soil; (b) three-layer soil .......... 7-10

Figure 7-3 (a) Coefficient $C$ in the interpretation formula for the relative density of the sand
layer over clay (Equation 7-7); (b) Coefficient $\beta$ in the interpretation formula for the relative
density of the sand layer embedded in clay (Equation 7-11). .............................................. 7-13
# LIST OF TABLES

Table 2-1. Summary of study cases in LDFE analysis .............................................................. 2-22

Table 2-2. Results of $H_k/D$ for all case studies ................................................................. 2-22

Table 3-1. List of all the soil parameters for Toyoura sand in the CSMC model ...................... 3-37

Table 3-2. Summary of the three miniature CPT tests in the centrifuge (Teh et al. 2010) ........ 3-37

Table 3-3. Parametric study of the centrifuge CPT in sand over clay soils ............................. 3-37

Table 4-1. Summary of the parametric study plan ................................................................. 4-41

Table 4-2. Summary of the key equations in the CSMC model ........................................... 4-41

Table 4-3. Values of the parameters in the CSMC model ..................................................... 4-42

Table 4-4. Summary of the results in the parametric study .................................................. 4-42

Table 5-1. Basic engineering properties of Toyoura sand and its parameters in the CSMC model .......................................................................................................................................... 5-38

Table 5-2. Summary of the parametric study plan ................................................................. 5-38

Table 5-3. Penetration depths for the penetration stages ....................................................... 5-38

Table 6-1. Summary of parametric study plan with cone pre-embedment depth ($d_{pre}$) ........ 6-21

Table 7-1. Summary of the equations to interpret the soil parameters ................................. 7-11
ACKNOWLEDGEMENTS

As the PhD journey approaches the end, I would like to thank so many people who have helped me to make it here.

An enormous thank you to my supervisors, Professor Yuxia Hu and Professor Mark Cassidy. With Yuxia, it has been eight years since we made the contact for the first time. You were the unit coordinator of Geomechanics (ENSC3009). It was your vivid lectures that opened the door for me to the fancy Geo-world in a scientific manner, although I did not really expect there was such a long ladder behind the door- a PhD study. Your patient guidance and optimistic attitude made the journey less stressful and even enjoyable. Nowhere else I can learn that much in both research and life except from you. A great thank you to Mark. You moved to Melbourne in my 2nd year, but the structure we settled has been the lighthouse for the whole time. Thanks for still making time for me in your intensive calendar there. You would never know how much we have missed your impressive laughter. A big thank you to Associate Professor Michael Zhou for helping with the program. It is not possible to finish the project without the right program being provided and modified.

A must-say thank you to Professor Liang Cheng, without your scholarship offer for the undergraduate study at UWA, everything I made in Australia would be impossible. I would also sincerely thank Professor Melinda Hodkiewicz, Associate Professor Andrew Abbo for leading me to the wonderful research world and helping me to build blocks to the PhD degree. The experiences of working with you are my very treasured memory in the lifetime. A special thank to April Kenny in Fluor Australia for the great mentorship you provided to me. You helped me understand the life value and my personal goals at the very right time.
I would thank Professor Fraser Bransby for the valuable discussion and the suggestion of always trying to understand the mechanism. I want to thank Professor Dong Wang and Associate Professor Jingbin Zheng for the help with learning Abaqus and valuable discussion on soil models.

I would also thank the administration team at the Centre for Offshore Foundation System and the Department of Civil, Environmental and Mining Engineering at UWA. Without your hard work, we would not sit in such a friendly and well-organized workplace.

I owe thanks to many friends who have made the PhD life memorable for me: Fuyu, Zefeng, Chao, Wangcheng, Tianqiang, Zhechen, Ci, Anamitra, Wenchao, Benya, Xuhao, and many other unnamed heroes. To all, we learn and grow all together, sometimes the former can be painful and challenging, but the latter is always enjoyable and well-deserved.

Last but not the least, thank you so much for my families. To my parents: you have supported me to peruse almost anything I want in my life without placing any restriction. It does not matter where we are as individuals right now and in the future, we are and will be always deeply connected. To myself: from the little boy hanging around construction sites to collect shiny stones to a PhD trained geotechnical engineer, you have made yourself!

Note: the thesis is being completed over the COVID-19 pandemic period. Many things we could never imagine before have become new normal. Lives have been lost and big disturbances have occurred to everyone. However, I believe each generation could have their own badges, being through the pandemic safely could be a shining badge for all of us! Best wishes to everyone.
AUTHORSHIP DECLARATION

This thesis is submitted as a series of papers that have been published or have been already for submission. The bibliographical details of the papers are presented below along with a description of the contribution from the candidate.

Paper 1 (presented in Chapter 2)


Estimated percentage contribution of the candidate is 65%. The candidate designed and performed the parametric study under the guidance of the co-authors. The candidate interpreted the results and proposed interpretation guidelines. The first draft was prepared by the candidate and the draft has been reviewed and amended by all the co-authors. The candidate has also presented the work in the OMAE conference 2019.

Paper 2 (presented in Chapter 3)

Xie, Qiang, Yuxia Hu, Mark J Cassidy and Mi Zhou. “Interpretation of CPT Data in Sand over Uniform Clay in a Geotechnical Centrifuge”, will be submitted to International Journal of Physical Modelling in Geotechnics.

Estimated percentage contribution of the candidate is 65%. The candidate designed and performed the parametric study under the guidance of the co-authors. The candidate interpreted the results and proposed interpretation guidelines. The first draft was prepared by the candidate and the draft has been reviewed and amended by all co-authors.

Paper 3 (presented in Chapter 4)
Xie, Qiang, Yuxia Hu, Mark J Cassidy and Mi Zhou. “Cone Penetration Test in a Thin Sand Layer Embedded in a Uniform Clay – LDFE Analysis”, will be submitted to International Journal of Geomechanics.

Estimated percentage contribution of the candidate is 65%. The candidate designed and performed the parametric study under the guidance of the co-authors. The candidate interpreted the results and proposed interpretation guidelines. The first draft was prepared by the candidate and the draft has been reviewed and amended by all co-authors.

**Paper 4 (presented in Chapter 5)**


Estimated percentage contribution of the candidate is 65%. The candidate designed and performed the parametric study under the guidance of co-authors. The candidate collected and interpreted the results. The first draft was prepared by the candidate and the draft has been reviewed and amended by all co-authors.

**Paper 5 (presented in Chapter 6)**


Estimated percentage contribution of the candidate is 65%. The candidate designed and performed the parametric study under the guidance of co-authors. The candidate collected and interpreted the results. The first draft was prepared by the candidate and the draft has been reviewed and amended by all co-authors.

xxiii
The candidate contributions have been approved by the co-authors of the papers.

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13/05/2020
1 INTRODUCTION

1.1 BACKGROUND

1.1.1 Why does the cone penetration test (CPT) need to be investigated?

The CPT is the most commonly used site investigation test worldwide. The standard cone penetrometer has a conical tip with a base area of 10 cm$^2$ (diameter = 3.57 cm) and 60° apex angle at the tip. As suggested by the International Reference Test Procedure (IRTP) and the American Society for Testing and Materials (ASTM) standards, the cone travels at a rate of 2 cm/s into the ground to make the drainage condition feasible to the assumption that the fully drained and fully undrained conditions prevail for clean sand and pure clay respectively (Kim et al., 2008). The cone tip resistance ($q_c$) is recorded by the sensors installed near its shoulder, along with the sensors installed around the cone shoulder and sleeve for the measurement of pore water pressure ($u$) and sleeve friction ($f_s$). Figure 1-1 illustrates the cone penetrometer with the three measurement components.

Due to its continuous penetration and good repeatability, CPT has been adopted in both onshore and offshore practices. Practice engineers use the measured CPT data to interpret the underground stratification and soil classification. Recent developed full-flow penetrometers (i.e. T bar and ball) have attracted increasing attention for their better profiling ability in soft sediments (Zhou et al. 2013). In stiffer soils (i.e. stiff clay or sand), the cavity formation during T-bar or ball penetrations can hinder the interpretation of soil parameters based on full-flow mechanisms. Hence, CPT is still a more preferred probe for site investigations in stiff soils (i.e. very stiff clay or sand). Various engineering properties of soils can be derived from the cone resistance profile. For example, the undrained shear strength of clay ($s_u$) can be obtained by normalizing the cone resistance by a bearing factor ($N_b$) and the relative density of sand calculated by empirical formulas that relate cone resistance to density (Lunne et al., 2002). The information and properties derived by the CPT
about the ground is crucial for engineering design and risk assessment of foundations and other geotechnical infrastructure. Therefore, any improvement on the interpretation of CPT data has the potential to benefit the geotechnical engineering community.

1.1.2 Why does CPT in layered soils need to be studied?

Soils are often deposited in layers during their long history of geological processes. In the offshore environment, where foundations are built for oil, gas and renewable energy infrastructure, layered seabeds are commonly encountered, particularly in emerging operating fields. Many such areas have been reported across the globe, such as (1) multilayered clay deposits in the Gulf of Thailand and Sunda Shelf (Castleberry and Prebaharan, 1985, Handidjaja et al., 2004, Kostelnik et al., 2007, Chan et al., 2008, InSafeJip, 2010) and (2) multilayered deposits with interbedded sand in the Gulf of Suez, Southeast Asia, Gulf of Mexico and offshore South America (Baglioni et al., 1982, Dutt and Ingram, 1984, Teh et al., 2009). Care needs to be taken for offshore engineering designs in layered soils, as engineering failures can occur when the stratification is poorly understood. One example of this is the punch-through failure of foundations in strong over soft soil conditions (Lu, 2008, Ullah et al., 2017).

Another example, where the understanding of layered soil is crucial, is soil liquefaction assessment. Liquefiable conditions are rarely found in uniform deposits but more often require liquefiable soil deposits to be overlain by less permeable soils. Reliable interpretations on the engineering properties of each soil layer serves as a crucial basis for liquefaction assessment.

Although the CPT has a long history of being utilized to detect soil stratifications, caution remains when using the measured profile to interpret soil parameters for each individual layer, especially where a thin layer is involved. The cone senses the impact from soils ahead and behind the cone by several cone diameters from the cone tip (Lunne et al., 2002). When the sensing zone expands across layer interfaces, the measured resistance not only represents the characteristics of the soil
layer where the cone is currently embedded, but also includes the impact from the layers above and/or below. The well-established guidelines on interpreting CPT data are based on CPTs in single layer soils. Applying such guidelines in layered soils directly can result in faulty interpretations (Tehrani et al., 2017). For example, the strength parameters of the layer could be underestimated when the layer is embedded in softer soils, while they could be overestimated when the layer is embedded in stiffer soils (Ma et al., 2016, Ma et al., 2017). Therefore, improving the understanding and interpretation of the CPT in layered soils is necessary and important.

With the advances in numerical methodology and computational capability, the whole process of the cone penetration and its measured cone resistance were modelled in this study. After verifying the numerical method against existing physical test data, comprehensive parametric studies were carried out for a CPT in different layered soil profiles. This investigated the soil-cone interactions during the continuous penetration of the cone through layered soils in order to establish guidelines to interpret the soil strength in each layer.

1.2 RESEARCH OBJECTIVES

There are three objectives in this thesis, which will be achieved in sequence.

1.2.1 Objective I

Provide guidelines to interpret CPT data in layered soil: based on the measured cone resistance profile formulas will be proposed to identify the layer interface(s) and to estimate the soil parameters of each layer. For clay and sand, the identified soil parameters are the undrained shear strength and the relative density respectively.

1.2.2 Objective II

Reveal flow mechanisms and provide insightful information on the soil response to the cone penetration test. The developments of soil stresses and soil displacements will be revealed around
the penetrating cone to understand the cone resistance profiles. Particular attention is offered when the cone penetrates across soil layer interfaces.

1.2.3 **Objective III**

Explore the efficiency of modelling the deep penetration of the cone in layered soils: various model setups are explored and compared to provide recommendations on modelling cone penetration problem in layered soils efficiently.

1.3 **THESIS OUTLINE**

The thesis is organized by five Papers 1 to 5, they are either published or ready for submission. Figure 1-2 illustrates five soil strata being studied, (a)-(d) being included in Papers 1 to 4; (c) and (e) being included in Paper 5. The soil strata range from two-layers (i.e. stiff over soft clay and sand over clay) to three-layer soils with both stress-dependent material (sand) and stress-independent material (clay) involved. Figure 1-3 summarizes the thesis structure that links the Chapters to the corresponding publications and the relevant objectives. A brief summary of each chapter is provided below.

- Chapter 2 studies a miniature cone penetration test in stiff over soft clay in centrifuge testing scenario (high g) by a numerical parametric study. The study aims to reveal the soil flow mechanism with the cone penetration and to provide guidelines to interpret CPT profiles for the top stiff clay layer.

- Chapter 3 studies a miniature cone penetration test in sand over clay in centrifuge testing scenario (high g). The study aims to provide insights in the soil flow mechanism and stress development with the cone penetration. A parametric study is conducted to establish interpretation framework for the CPT profiles in the soil strata.

- Chapter 4 studies a standard cone (cone diameter = 0.0357 m) penetration test in a thin sand layer embedded in clays. The study aims to reveal the soil flow mechanism and stress
development with the cone penetration. A parametric study is conducted to propose formulas for interpretation purpose.

- Chapter 5 studies a standard cone (cone diameter = 0.0357 m) penetration test in a thin sand layer sandwiched between different clay layers (i.e. different undrained shear strengths). The study investigates the flow mechanism and stress development for various clay layer combination (stiff clay on top and soft clay at bottom; soft clay on top and stiff clay at bottom). A parametric study is conducted to propose guidelines on the CPT data interpretation.

- Chapter 6 investigates different methodologies to model the CPT problem in soft-stiff-soft soils: (i) small strain analysis for pre-embedded cones at various depths; (ii) large deformation finite element (LDFE) analysis for the cone penetrating from soil surface; and (iii) to conduct LDFE analysis for cones pre-embedded at various depths. Comparison of their results is made to search for ways to save computation cost for numerical modelling probe penetration problem.

- Chapter 7 summarizes the main findings of the thesis and their practical implications and provides the recommendation to future works.

1.4 REFERENCE


Introduction

Figure 1-1 Schematic of CPT

Figure 1-2. Soil strata in this study

(a) Stiff clay
    Soft clay
(b) Sand
    Clay
(c) Clay A
    Sand
    Clay A
(d) Clay A
    Sand
    Clay B
(e) Soft clay
    Stiff clay
    Soft clay
<table>
<thead>
<tr>
<th>Objective I</th>
<th>Objective II</th>
<th>Objective III</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chapter 2</td>
<td>Journal Paper 2: <em>Interpretation of CPT Data in Sand over Uniform Clay in a Geotechnical Centrifuge</em></td>
<td></td>
</tr>
<tr>
<td>Chapter 3</td>
<td>Journal Paper 3: <em>Cone Penetration Test in a Thin Sand Layer Embedded in a Uniform Clay – LDFE Analysis</em></td>
<td></td>
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<tr>
<td>Chapter 4</td>
<td>Journal Paper 4: <em>Cone Penetration Test in a Thin Sand Layer Sandwiched by Different Clay Layers</em></td>
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</tr>
<tr>
<td>Chapter 5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Chapter 6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Chapter 7: <em>Conclusion</em></td>
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<td></td>
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</tbody>
</table>

Figure 1-3. Thesis structure and publication summary
2 LDFE ANALYSIS ON CONE PENETRATION TEST IN STIFF OVER SOFT CLAY FOR CENTRIFUGE TEST

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ABSTRACT

This paper describes a numerical study on soil characterization of stiff over soft clays in centrifuge tests using cone penetration test (CPT), especially when the top stiff layer is thin relative to the centrifuge cone size. An extensive parametric study was conducted using the Large Deformation Finite Element (LDFE) analysis, with the cone penetrating continuously from the soil surface. The LDFE model has been validated against existing physical test data with very good agreement. Since the bottom soft clay was normally thick enough to fully mobilize the ultimate cone resistance, its undrained shear strength can be interpreted by the existing approach for cone deep penetration in a uniform clay layer. Thus, the challenge was to interpret the strength of the top stiff layer, where the layer thickness was not thick enough to fully mobilize its ultimate resistance. Both top layer thickness ratios (to the cone diameter) and layer strength ratios were considered in the parametric study. Based on the results from LDFE analyses, the interpretation formula of the undrained shear strength in the top stiff layer was proposed as a set of new bearing factors. The proposed cone bearing factor was a function of the ratio of the measured peak cone resistance in the top layer to the stable/ultimate cone resistance in the bottom layer and the ratio between the top layer thickness to the cone diameter. The formula can be used directly when the top layer thickness was known based on the sample preparation. However, the layer interface can be identified based on the study
here, if the top layer thickness was not certain. A design flow chart was provided for the interpretation of top clay layer strength and top layer thickness based on the cone resistance profile obtained from the CPT test.

KEYWORDS

Cone Penetration test, layered soil, centrifuge test, LDFE analysis
2.1 INTRODUCTION

2.1.1 Background

Offshore oil and gas exploration is moving towards deeper and undeveloped environments due to the depletion of the known reserves in shallow waters in traditional hydrocarbon regions. Layered soils have been encountered at a high frequency in such areas. For example, a Joint Industry Funded Project (Osborne et al., 2009), covering many popular oil and gas exploration areas (e.g. The Sunda Shelf, offshore Malaysia, Australia’s North-West Shelf, South China Sea), reports that over 75% of its case study datasets involve stratified seabed profiles. The layered deposits are also encountered in the Gulf of Mexico and Gulf of Guinea (Menzies and Roper, 2008, Colliat and Colliard, 2010, Menzies and Lopez, 2011).

To study the behaviour of offshore foundations in layered soils, centrifuge tests are often conducted (Teh et al., 2010, Hu et al., 2013, Ullah et al., 2017) due to its ability to model the dimensions of typical offshore foundations (e.g. spudcan) under in situ stress levels. The proper characterization of soil layers in the test is the basis to analyse the foundation (e.g. spudcan) response profiles obtained in the tests. The soil characterisation includes two aspects: (1) the thickness ratio of the soil layers relative to the structure diameter and (2) the strength ratio between the two successive soil layers, since these two ratios are the main factors that affect the foundation behaviour in layered soils.

To quantify the characteristics of layered soils, CPT is the most commonly used in-situ investigation test for both onshore and offshore site investigations. It is also a commonly used probe in laboratory tests, including centrifuge model tests (Teh et al., 2010).

Due to the high acceleration level (i.e. high g) in centrifuge test, the different cone sizes used in the tests can have large prototype cone diameters. With the limited space (or sample height) in the strong box, when stiff-over-soft clay samples are prepared, the top layer thickness can be relatively thin compared to the cone diameter. This can make the interpretation of the soil strength in the top
stiff layer rather difficult. Although many formulas are available to interpret soil strength using CPT data with deep penetration, the relative thin top stiff clay layer cannot fully mobilize the cone resistance. The conventional interpretation formulas cannot be used.

2.1.2 Previous work

There were a number of studies conducted on cone penetration in single-layer clay with uniform shear strength using analytical, numerical and experimental methods. The bearing capacity factor of the cone has been extensively studied by the strain path method, hybrid strain path method, cavity expansion method and conventional small strain finite element analysis (Baligh, 1985, Teh and Houlsby, 1991, Yu, 2013). Recently, the methods of large deformation finite element analysis and centrifuge tests have been applied to study the problem (Van den Berg, 1994, Bolton et al., 1999, Lu et al., 2004, Walker and Yu, 2006, Liyanapathirana, 2009). Despite these efforts, the conventional practice is still to directly evaluate the in-situ shear strength ($s_u$) for clay by a constant cone bearing capacity factor, $N_b$, as $s_u = q_{net}/N_b$ in-situ. The cone factor can be obtained by correlating the net cone tip resistance to the shear strength measured from element tests conducted in laboratory (Chan et al., 2008, Ozkul et al., 2013), as $N_b = q_{net}/s_u$ in laboratory. Hence, once $N_b$ is known, the soil strength can be calculated by cone penetration resistance profile. However, based on typical laboratory tests (such as triaxial test and simple shear test) data, $N_b$ can range from 7.2 to 18 (Low et al., 2010, Low et al., 2011).

In contrast to the large amount of studies on CPT in single layer soils, there has been relatively little research on the effect of soil layering on CPT measurement. A small number of experimental tests (Treadwell, 1976, Silva and Bolton, 2004, Xu, 2007) provided the observation of the transition through soil layers. Numerically, Berg et al. (Berg et al., 1996), Ahmadi et al. (Ahmadi et al., 2005), Xu and Lehane (2008) and Walker and Yu (2010) studied the layering effects and influence zone around soil interfaces. The first analytical solution for cone penetration in layered soils was proposed by Vreugdenhil et al. (Vreugdenhil et al., 1994) using elasticity theory.
In brief, Berg et al. (Berg et al., 1996) presented an Eulerian analysis method of cone penetration in multilayer soils which is characterized by a non-associated Drucker-Prager criterion. Vreugdenhil et al. (Vreugdenhil et al., 1994) suggested correction factors to increase the insufficiently mobilized cone resistance in a thin layer, the factors are a function of the layer thickness ratio and layer strength ratio. However, for the interpretation of thin interbedded sand layer, lower correction factors were recommended for conservative interpretation of field test data. (Robertson and Fear, 1995, Youd et al., 2001, Ahmadi et al., 2005). Analytical solutions, i.e. cavity expansion, were employed to analyse the problem and a correction factor for the thin layer was proposed that fitted well with the reported field data (Mo et al., 2016). All these researchers, except Vreugdenhil et al. (Vreugdenhil et al., 1994), studied the layering effect for sediments with either surface or interbedded sand layers. More recently, a study was conducted for a CPT in many alternating sand and clay layers by physical tests [31]. Practical implementation suggestions were provided but no interpretation method was proposed (Van der Linden et al., 2018).

For cone penetration in multilayer clays, Walker and Yu (2010) studied the cone penetration in two-layer stiff-soft and three-layer stiff-soft-stiff layer by LDFE analysis in the FE package Abaqus/Explicit. The characteristics of cone resistance profiles during its continuous penetration into different layers were investigated, and the soil deformation around the cone tip provided better understanding of the soil failure mechanisms around the cone. However, there was no interpretation guideline provided in the study. Recently, numerical studies were conducted by LDFE analysis on soft-stiff-soft layering system in AFENA (Ma et al., 2014, Ma et al., 2015). These studies conducted comprehensive parametric analysis on variables of shear strengths and layering soil profiles. They provided a guideline on interpreting the undrained shear strength based on cone resistance profiles. The key findings can be summarized as following: (1) The layer interface can be identified at 0.8D below the kink of the resistance profile in the soft layer for the cone passing the soft-to-stiff interface, while 1.3D above the kink in the soft layer for the cone passing the stiff-to-soft interface, when the middle stiff layer was relatively thick (i.e. 15D – 20D); (2) The
influencing distance to sense the upcoming stiff middle layer was a function of the normalized middle layer thickness as $4 < t/D < 40$; (3) Formulas have been proposed for the cone factors in the top soft layer and correction coefficients to the cone factors for the middle stiff layer when the middle layer was not sufficiently thick to mobilize the ultimate cone resistance; (4) A flow chart was established to interpret soil layer boundaries and the undrained shear strengths in each identified layer. Moreover, the same researchers above also reported their investigations on cone roughness effect on layer interface identification, a new set of formulas were provided based on their results (Ma et al., 2017).

2.1.3 Objective of present work

This paper investigates soil characterisation in centrifuge model tests using CPT in stiff over soft clays using the LDFE analysis, where the mini cone used in centrifuge can have a large diameter in prototype dimensions due to high g-level. In the limited depth of the strong box, this means the ratio of the top layer thickness to the cone diameter can be so low that the ultimate cone penetration resistance can’t be reached. The LDFE analysis will be validated against centrifuge test data. Then a parametric study will be performed considering both top stiff layer thickness ratios to the cone diameter and the clay layer strength ratios within the range of centrifuge tests. Soil flow mechanisms will be revealed, and a framework to interpret soil layer interface and soil strength in each layer will be established for centrifuge modellers.

2.2 NUMERICAL ANALYSIS

2.2.1 Geometry and parameters

This paper studies a cylindrical cone penetrometer with a diameter $D$ with an apex angle $60^\circ$, penetrating into stiff over soft clays as illustrated in Figure 2-1. The top stiff layer has an undrained shear strength of $s_{ut}$, an unit weight of $\gamma_t$ and a thickness of $H$, and the bottom soft layer has an undrained shear strength of $s_{ub}$, an unit weight of $\gamma_b$ and nominally infinite thickness. Thus both
layers have uniform strengths. As the miniature cone used in the centrifuge is made of either stainless steel or aluminium with a polished surface, a fully smooth cone was simulated.

In centrifuge model test, the strength of a stiff clay sample ranges from 40 kPa to 120 kPa, and the strength of a soft clay sample ranges from 10 kPa to 40 kPa (Ma et al., 2015). The typical diameters of a centrifuge cone are 6 mm, 7 mm and 10 mm, where 10 mm cone is most common for centrifuges at the University of Western Australia. In a single layer of uniform clay, the cone resistance reaches its steady state when its penetration is deep (i.e. \( d > 12D \), where \( d \) is cone penetration depth). The centrifuge facility is commonly operated at 50 to 200 g. Therefore, in order to cover the practical range in centrifuge tests, the strength ratio of \( s_{\text{sat}}/s_{\text{sub}} \) ranging 2-10 and the top stiff layer thickness ratio ranging 2-16 are considered in the parametric study. A summary of the selected parameters are listed in Table 2-1.

### 2.2.2 LDFE analysis

FE analysis was performed using the FE package AFENA developed by Carter and Balaam (1995) at the University of Sydney. To study the continuous penetration of the cone, large deformation finite-element analysis incorporating the remeshing and interpolation technique with small strain (RITSS) technique was employed, which was implemented in the AFENA package by Hu and Randolph (1998a). The RITSS method can be categorised into an arbitrary Lagrangian-Eulerian (ALE) FE method, whereby a series of small strain incremental analyses is combined with automatic remeshing of the whole domain, followed by interpolation of field variables from the old mesh to the new mesh. To optimize the mesh after each remeshing step, H-adaptive mesh refinement cycles (Hu and Randolph, 1998b) were implemented to minimize discretization errors. In this paper, the displacement increment and the remeshing steps are chosen such that the cumulative penetration between remeshing stages remained in the small strain range and is less than half of the minimum element size.
The simulation of the cone penetration started with the cone tip embedded in soil and the cone shoulder at the soil surface. Before starting the cone penetration in layered soils, initial in-situ stresses were created in the soil domain. The cone penetration depth reported in this paper is referencing the cone shoulder point as illustrated in Figure 2-1. An axisymmetric soil domain was set as 100D in radius and 100D in depth to avoid domain boundary effects. Hinge and roller conditions were applied along the base and vertical side of the soil domain. Six-noded triangular elements with three internal Gauss points were used. Figure 2-2 illustrates a typical mesh setup at the start of the cone penetration analysis. The cone is modelled as a rigid body. This is achieved by defining all the boundary nodes of the cone and all the nodes are displaced at the same rate (i.e. displacement controlled analysis).

The clay was modelled as a linear elastic-perfectly plastic material obeying a Tresca yield criterion. The two elastic parameters for the model are Young’s modulus (E) and Poisson’s ratio (v). The plastic parameter is the undrained shear strength (s_u) defining the size of the yield surface. The elastic parameters are considered independent of the stress conditions, hence a constant values was adopted with a constant stiffness ratio of E/s_u = 500 for all the analyses. The effect of the variation of E/s_u on cone resistance in layered soil has been systematically explored by Ma et al. (2017). The cone penetration rate was set high enough to reach an undrained condition in the centrifuge test. For undrained clays, the Poisson ratio (v) in LDFE/RITSS analysis is set to v = 0.49 (i.e. close to 0.5) to give minimal volumetric strains while maintaining numerical stability.

2.2.3 LDFE model validation

The LDFE result was validated against centrifuge test data. The centrifuge test was operated at 50g for an 11.29 mm diameter model cone in a two-layer clay at the Technical University of Denmark. After the tests, the water content of each clay layer was measured. The undrained shear strength was then calculated based on the overconsolidation ratio and the water content of the sample (Koumoto and Houlsey, 2001). The undrained shear strength at the top layer was 26 kPa while the
shear strength at the bottom layer is 6 kPa (see Group I in Table 2-1). To verify the numerical model, the centrifuge test was simulating the cone test in the prototype dimension, with a cone diameter of 0.5645 m (i.e. 11.29 mm×50 = 0.5645 m) and top layer thickness of 2.7m. Shear strength of each layer was assigned as the measurement (i.e. $s_{ut} = 26$ kPa and $s_{ub} = 6$ kPa). Figure 2-3 shows the comparison of the numerical results with the experimental data. It is clearly shown that the numerical model works very well. For the standard cone used in site investigations (i.e. $D = 0.0357$ m) of single and double layer clays, the LDFE/RITSS model was validated against site investigation data (Ma et al., 2014, Ma et al., 2015). The difference of cone performance in centrifuge and in situ is due to the cone diameter. As the in situ cone diameter is relatively small, the cone resistance can reach its steady state fairly easily with relatively high $H/D$. However, in centrifuge tests, $H/D$ is relatively low, as $D$ is quite large in prototype, the cone resistance cannot reach its steady state in a relatively thin clay layer. This will make the soil strength interpretation erroneous. Hence, this study can provide new interpretation formulas for centrifuge tests.

### 2.3 RESULTS AND DISCUSSION

#### 2.3.1 Soil failure mechanisms

Soil flow mechanisms and resistance profiles during cone penetration process are linked directly, as shown in Figure 2-4 and Figure 2-5. Figure 2-4 shows a typical resistance profile with $s_{ut}/s_{ub} = 4$ and $H/D = 4$. Cone penetration resistances at six discrete penetration depths (i.e. six Stages of $A_1$ to $A_6$) are marked on the profile in Figure 2-4. Figure 2-5 depicts soil flow mechanisms corresponding to these penetration depths.

At Stage $A_1$ of $d/D = 0.8$ the soil around the cone tip is being pushed both vertically and laterally in the outward direction, while the soil around the cone shoulder flows upwards, leading to soil heave to the surface (Figure 2-5 (a)). When the cone penetrates to Stage $A_2$ of $d/D = 1.2$ the soil flow is concentrated in a limited zone beneath the cone in the top stiff layer in Figure 2-5 (b). At this stage, the once reaches its peak resistance in Figure 2-4. After this Stage, the cone starts to
sense the bottom soft layer and the cone resistance starts to drop sharply. Once the cone tip enters the bottom soft layer at Stage A3 of d/D = 3.5, the cone shoulder is still in the top stiff layer, soil flow is predominantly attracted to the underlying soft clay layer (see Figure 2-5 (c)), leading to a bending of the layer interface. When the cone is passing through the layer interface to Stage A4 of d/D = 4.5, the cone penetration resistance continues to drop sharply (i.e. Stages A3 to A4 in Figure 2-4). Finally, when the full cone tip passes the deformed top stiff layer, and is fully embedded in the bottom soft layer (Figure 2-5 (e) and (f)), the soil flow is confined in the bottom soft layer.

The cone resistance reaches the peak in top stiff layer and stabilizes in bottom soft layer when the cone is fully restricted in the soil layer where it is embedded (A2 and A3). There is no soil being trapped down from the top stiff layer to the bottom soft layer by the cone.

2.3.2 Interpretation of shear strength of the top-stiff layer

The effect of the absolute soil strength on the cone resistance profile is studied by keeping the soil layer strength ratio constant at $s_{ut}/s_{ub} = 4$, but varying the absolute bottom layer strength, $s_{ub} = 10$ kPa and 20 kPa (Group III in Table 2-1). The effect of the absolute top layer thickness is explored by keeping the normalized top layer thickness and the layer strength ratio constant at $H/D = 4$ and $s_{ut}/s_{ub} = 4$ but varying the absolute top layer thickness (Group IV in Table 2-1). It is found that, although the cone resistance profiles ($q_{net}$) are affected by the absolute $s_{ub}$ and $H$, the normalized cone resistance ($q_{net}/s_{ub}$) profiles are not affected. Hence, the soil strength ratio ($s_{ut}/s_{ub}$) and the normalized top layer thickness ($H/D$) are the two factors that influence the normalized cone resistance profiles. Therefore, these two factors are studied further in the following Sections.

2.3.2.1 Effects of top layer thickness ratio and soil strength ratio

The effects of the normalized top layer thickness ($H/D$) and soil layer strength ratio ($s_{ut}/s_{ub}$) on cone resistance profiles are shown in Figure 2-6. The top layer thickness ratio varies from $H/D = 2$ to 12 at constant strength ratio of $s_{ut}/s_{ub} = 4$ (Group V in Table 2-1); and the soil layer strength ratio varies from $s_{ut}/s_{ub} = 2$ to 6 at constant normalized top layer thickness of $H/D = 4$. It is apparent that
the stable cone resistances in the bottom soft layer are not affected by the top stiff layer thickness and the strength ratio of the layers. This can also been seen in Figure 2-5 (f) as the soil flow mechanism is confined to the bottom layer without the influence from the top layer. However, the cone peak resistance in the top stiff layer varies with both factors of H/D and s_{ut}/s_{ub}.

2.3.2.2 Proposed interpretation methodology

To establish a framework for interpreting the shear strength of the top stiff layer, a cone bearing factor is proposed based on the peak cone resistance (q_p) in the top layer and its soil strength (s_{ut}). The proposed cone bearing factor is expressed as N_{bq} in Equation 2-1,

\[ N_{bq} = \frac{q_p}{s_{ut}} \]  

Equation 2-1

Based on the numerical results for the Groups V, VI and VII in Table 2-1, the cone bearing resistance factor (N_{bq}) is a function of q_p/q_b and H/D. The best fitted equation can be expressed as shown in Equation 2-2 with R^2 = 0.996.

\[ N_{bp} = -0.6 \ln \left( \frac{q_p}{q_b} \right) + 2.8 \ln \left( \frac{H}{D} \right) + 5.44 \]  

Equation 2-2

Once the cone bearing factor is obtained by Equation 2-2, the top layer soil strength can be interpreted using the cone peak resistance as shown in Equation 2-3 below.

\[ s_{ut} = \frac{q_p}{N_{bp}} \]  

Equation 2-3

2.3.2.3 Identification of layer interface

In most cases in centrifuge testing, the top layer thickness is known during test design. However, due to the handling of the sample during sample preparation, it would be beneficial that the layer interface can also be identified/confirmed based on the CPT resistance profile. Figure 2-7 shows the geometry of one typical resistance profile where the thickness of the stiff layer could be calculated by subtracting the distance between the kink point and layer interface (H_k/D) from the
measured total distance from the top surface to the kink (\(H_t/D\)), i.e. \(H/D = H_t/D - H_k/D\). Since \(H_t/D\) can be measured from the cone resistance profile directly, the identification of the layer interface is down to finding out \(H_k/D\).

From Figure 2-6, it can be seen that \(H_k\) is a function of both \(H/D\) and \(s_{\text{ut}}/s_{\text{ub}}\). Table 2-2 summarizes the results of \(H_k/D\) from all cases studied in the parametric analysis (see all Groups in Table 2-1). It shows that, for lower top layer thickness ratio of \(H/D < 8\), \(H_k/D\) is a function of both ratios of \(H/D\) and \(s_{\text{ut}}/s_{\text{ub}}\). However, for higher top layer thickness ratio of \(H/D \geq 8\), \(H_k/D\) is only related to \(s_{\text{ut}}/s_{\text{ub}}\).

However, when the top layer thickness is unknown, \(H_k/D\) needs to be estimated based on the known cone resistance profile only. As \(H_t/D\) and \(q_{\text{p}}/q_{\text{b}}\) can be measured directly, these two variables can be used for layer interface identification, i.e. \(H_k/D\) estimation.

To estimate \(H_k/D\) based on the measured \(H_t/D\) and \(q_{\text{p}}/q_{\text{b}}\), the critical \(H_t\), i.e. \(H_c\), needs to be defined first. For all cone resistance profiles, \(H_c\) is defined as, for all resistance profiles with constant \(q_{\text{p}}/q_{\text{b}}\), but different \(H/D\), \(H_k/D\) increases with increasing \(H/D\) when \(H_t < H_c\); \(H_k/D\) becomes constant when \(H_t > H_c\). By collating all LDFE results of all case studies, the \(H_c/D\) at various \(q_{\text{p}}/q_{\text{b}}\) are plotted in Figure 2-8. The fitted curve can be expressed in Equation 2-4 with \(R^2 = 0.9633\)

\[
\frac{H_c}{D} = 0.22 \ln \left( \frac{q_{\text{p}}}{q_{\text{b}}} \right) + 8.31 \quad \text{Equation 2-4}
\]

After finding \(H_c/D\), the soil layer interface of \(H_k/D\) can be estimated based on the curve fitting of the LDFE results under two scenarios:

(a) When \(H_t/D < H_c/D\), \(H_k/D\) is a function of \(H_t/D\) and \(q_{\text{p}}/q_{\text{b}}\) and can be expressed as Equation 2-5 with \(R^2 = 0.984\),

\[
\frac{H_k}{D} = 0.42 \ln \left( \frac{q_{\text{p}}}{q_{\text{b}}} \right) + 0.19 \ln \left( \frac{H_t}{D} \right) + 0.2 \quad \text{Equation 2-5}
\]
(b) When $H_t/D \geq H_c/D$, $H_k/D$ is a function of only $q_p/q_b$ and can be fitted in Equation 2-6 with $R^2 = 0.9633$, shown in Figure 2-9.

$$\frac{H_k}{D} = 0.22 \ln \left( \frac{q_p}{q_b} \right) + 0.81$$  \hspace{1cm} \text{Equation 2-6}

### 2.4 INTERPRETATION FRAMEWORK

Based on the above analysis, both shear strength and layer thickness of the top stiff layer can be obtained through the cone resistance profiles. As the bottom layer thickness should be always large enough to develop the ultimate resistance which can be interpreted by conventional interpretation methods, there is no need to set additional guidelines for the bottom layer. Figure 2-10 provides the interpretation framework for $s_{ut}$ and $H_k$.

### 2.5 CONCLUSION

The paper reports the results from LDFE analysis using the state-of-the-art RITSS method, simulating continuous penetration of a mini-cone penetration in soil sample in a centrifuge test in prototype dimensions. The detailed flow mechanism was revealed regarding to the layer interface deformation and the effect of new layer to the soil movement around the penetration cone. The extensive parametric study on the stiff over soft clay layers covered a range of normalized soil properties and layer geometries. Based on the analysis results, a formula was proposed to interpret the undrained shear strength of the top stiff layer using the measured cone resistance directly, provided the thickness of the top stiff layer was known. However, if the thickness of the top stiff layer is unknown (or uncertain), a framework was established to identify the layer interface location (i.e. the thickness of the top layer). As the formulas are proposed based on the planned ranges of soil layer profiles and parameters, any interpolation outside of the ranges needs to be conducted with caution.

Current study has been focusing on uniform layers of soils. The study of centrifuge cone penetrating into stiff over NC clay layers is underway.
2.6 REFERENCES


Figure 2-1. Schematic diagram of cone penetration in stiff over soft clays

Figure 2-2. Mesh setup of cone in two-layer clay
Figure 2-3. Validation against centrifuge test

Figure 2-4. Resistance profile with marked stages (H/D = 4, s\text{uf}/s\text{ub} = 4, base case in Table 2-1)
Figure 2-5. Soil flow mechanisms at different penetration stages
Figure 2-6. Resistance profiles for (a) various top layer thickness ratio (H/D) at $s_{ut}/s_{ub} = 4$; (b) various soil layer strength ratio ($s_{ut}/s_{ub}$) at H/D = 4

Figure 2-7. Resistance profile with defined variables for interpretation purpose
Figure 2-8. Relationship between critical depth $H_c/D$ and $q_p/q_b$

Figure 2-9. Relationship between $H_k/D$ and $q_p/q_b$ for $H_t/D \geq H_c/D$
Figure 2-10. Interpretation framework of $s_{st}$ and $H$
Table 2-1. Summary of study cases in LDFE analysis

<table>
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<th>Group</th>
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<th>Purpose</th>
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<tr>
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<td>4.33</td>
<td>LDFE model Validation</td>
</tr>
<tr>
<td>II</td>
<td>4</td>
<td>4</td>
<td>Base case (su=10 kPa and D=1 m)</td>
</tr>
<tr>
<td>III</td>
<td>4</td>
<td>4</td>
<td>Study the effect of absolute shear strengths (su1=10 kPa and su2=20 kPa)</td>
</tr>
<tr>
<td>IV</td>
<td>4</td>
<td>4</td>
<td>Study the effect of absolute top layer thickness (H1=1 m and H2=2 m)</td>
</tr>
<tr>
<td>V</td>
<td>2;4;6;8;10;12;14;16</td>
<td>4</td>
<td>Study the effect of normalized top layer thickness</td>
</tr>
<tr>
<td>VI</td>
<td>4</td>
<td>2;4;6;8;10</td>
<td>Study the effect of soil strength ratio</td>
</tr>
<tr>
<td>VII</td>
<td>2;6;8;10;12;14;16</td>
<td>2;6;8;10</td>
<td>Full range of parametric study for establishing interpretation framework</td>
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Table 2-2. Results of Hk/D for all case studies

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<th>8</th>
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</table>
3 INTERPRETATION OF CPT DATA IN SAND OVER UNIFORM CLAY IN A GEOTECHNICAL CENTRIFUGE- LDFE ANALYSIS

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ABSTRACT

This paper presents the results of numerical investigations on the cone penetration tests (CPTs) in sand over clay soils in centrifuge test scenarios by the large deformation finite element (LDFE) analysis method. Due to the high accelerations and limited strongbox dimensions, the interpretation of the CPT data of sand over clay in centrifuge tests needs to be established because the conventional interpretation methods are for a single layer of sand is no longer applicable. In the LDFE analysis, the critical state Mohr-Coulomb (CSMC) model for sand was deployed to capture its stress-dependent response under drained conditions. A linear elastic-perfect plastic model was used for clay under undrained conditions. After the validation of the LDFE method with existing centrifuge test data, a parametric study was conducted. A typical CPT test profile is divided into six stages from shallow penetration at the sand surface to deep penetration into the bottom clay layer. The development of the soil flow mechanism and stress bubbles are displayed at these six stages. The cone peak resistance in the sand was reached when the cone sensed the bottom soft clay. After reaching the peak resistance, double shear planes were formed as the cone approached the bottom clay. The shear planes disappeared when the cone was fully embedded in the bottom clay. Based on the comprehensive parametric study, the cone peak resistance, the peak location and the kink distance in the clay were found to be functions of the sand relative density, sand layer
thickness and shear strength of the clay layer. Based on the LDFE results, formulas are proposed to identify the relative density of the top sand layer and the sand-clay layer interface. The formulas can be used to interpret CPT data in centrifuge tests with sand-over-clay soil sample. Compared with existing formulas derived for single layer soil, the proposed interpretation framework worked better.

**KEYWORDS**

CPT test, centrifuge test, critical state soil model, large deformation FE analysis, layered soil
3.1 INTRODUCTION

3.1.1 Background

Layered soils are commonly encountered in emerging areas for offshore explorations. Many such areas have been reported across the globe, such as (1) multilayered clay deposits in the Gulf of Thailand and Sunda Shelf (Castleberry and Prebaharan, 1985, Handidjaja et al., 2004, Kostelnik et al., 2007, Chan et al., 2008, InSafeJip, 2010) and (2) multilayered deposits with interbedded sand in the Gulf of Suez, Southeast Asia, Gulf of Mexico and offshore South America (Baglioni et al., 1982, Dutt and Ingram, 1984, Teh et al., 2009). Correct identification of the soil stratigraphy and soil properties for each layer is crucial for offshore foundation design and geohazard assessment (e.g., punch-through failure).

Centrifuge testing is an effective methodology in geotechnical engineering for investigating the behaviours of offshore structures in layered soils. It has been successfully used to study offshore structure (e.g., spudcan) penetration problems in multilayered soils, such as layered clay (Hossain and Randolph, 2010), sand over clay (Teh et al., 2010) and clay-sand-clay soils (Ullah et al., 2017). It is capable of modelling full-scale structures under field stress levels and strength gradients with small-scale models (Mo et al., 2015). Therefore, geotechnical centrifuge tests have become a routine test for investigating large offshore structure foundations interacting with soils.

Due to its reliability and repeatability, the cone penetration test (CPT) has gained wide acceptance for soil characterization (Lunne et al., 2002). Data collected during the CPT method include the cone tip resistance, $q_c$, and sleeve friction, $f_s$. The piezocone penetration test (CPTu), a modified CPT with an added water pressure sensor, can measure the pore pressure at the shoulder of the cone, $u_2$, to provide additional information.

For geotechnical centrifuge tests, a series of in-flight miniature CPTs were performed to evaluate the soil characteristics of each layer for sand over clay prior to the model structures (e.g., spudcan) penetration test being conducted (Teh et al., 2010). Results showed that the CPT peak resistance in
the sand layer was significantly affected by the sand layer thickness, relative density and centrifuge gravity level (i.e., g-level), which made the interpretation of the CPT data inconclusive. Thus, rather than relying on the CPT data, the measured sand relative density ($I_D$) was used during the preparation of the sample and to estimate the undrained shear strength ($s_u$) of the clay based on the vertical effective stress ($\sigma'_v$) of clay.

### 3.1.2 Previous works

Many studies have been conducted on the CPT in single layer clay to understand its penetration mechanism and provide guidelines for interpreting the shear strength parameters. The undrained shear strength, $s_u$, can be derived by

$$s_u = \frac{q_c - \sigma'_v}{N_b} \quad \text{Equation 3-1}$$

where $\sigma'_v$ is the overburden pressure at the cone shoulder and $N_b$ is the bearing capacity factor (cone factor in some studies). Appropriate values for this cone factor have been extensively derived using different techniques, such as the strain path method, hybrid strain path method, cavity expansion method and conventional small strain finite element (FE) analysis (Baligh, 1985, Teh and Houlsby, 1991, Yu, 2013). As numerical techniques have advanced, large deformation finite element (LDFE) analysis has been applied to study such problems (van den Berg, 1994, Lu et al., 2004, Liyanapathirana, 2009). LDFE analysis gives more rigorous solutions compared with small strain finite element analysis, since the latter cannot generate the residual stress field around the cone (Houlsby et al., 1985). An agreed upon conclusion from these analyses is that the cone factor is a linear function of the logarithm of the rigidity index, cone roughness and stress anisotropy factor. Different studies have proposed different values for the coefficient as a linear function. By assembling high-quality databases worldwide, Low et al. (2010) reported an average value of 13.5 for lightly overconsolidated clays.
For CPT in sandy deposits, although it has been repeatedly reported that the strength of cohesionless soil is too complicated to be represented by only the relative density, the concept of the relative density is still commonly used by many practising engineers (Lunne et al., 2002). The complexity is introduced since the cone resistance in cohesionless soil is a function of not only the soil density but also other factors, such as the in situ effective stress and compressibility. The chamber calibration (CC) test is heavily employed to establish the relationship between the relative density, in situ stress and measured cone resistance. Many correlations between the cone resistance and sand relative density have been provided by CC tests (Baldi et al., 1981, Chapman and Donald, 1981, Villet and Mitchell, 1981). However, there is no unique relationship across all types of sand (Robertson and Campanella, 1983).

In contrast to the extensive studies with the CPT in single layer soils, there has been relatively little research on the layering effect on the CPT results. Physically, a small number of researchers have observed the transition through soil layer interfaces in the resistance profiles in their tests (Treadwell, 1976, Silva and Bolton, 2004, Xu, 2007). Researchers have numerically studied the layering effect on the cone resistance profile and cone influence zone near the soil layer interface (Van den Berg et al., 1996, Ahmadi et al., 2005, Xu and Lehane, 2008, Walker and Yu, 2010, Ma, 2016, Mo et al., 2015). The first analytical solution was proposed by Vreugdenhil et al. (1994) using elasticity theory.

Key findings for the layer effects are summarized here: (1) the cone resistance is not only determined by the soil at its location, but the cone can sense the impact from the soil several diameters ahead and behind the cone tip. (2) The size of the influence zone is a function of various factors, such as the soil stiffness difference between the layer interface or the soil density. (3) The cone is not able to achieve its ultimate resistance when the soil layer is not sufficiently thick. The correction factor needs to be applied as a multiplier for the measured cone resistance before the data can be used for soil characteristic interpretation. (4) Some research has proposed functions for
calculating the correction factor as a function of the layer thickness and the stiffness ratio between two adjacent layers (Vreugdenhil et al., 1994, Ahmadi et al., 2005). Youd and Idriss (2001) suggested a lower bound to the correction factors for a conservative interpretation based on an extensive analysis of field data.

Teh et al. (2006) performed a series of miniature piezocone tests in sand over normal consolidated (NC) clay in a centrifuge test under an acceleration of 100 g. They found that the cone resistance profile varied substantially with the top layer thickness, which highlighted the effect of the influence zone. The influence zone ahead and behind a penetrating cone was examined in their study. The key conclusions can be summarized as follows: (1) the effect of the top layer thickness is greater than the effect of the relative density on the cone resistance; (2) the influenced zone in sand (of \(I_D = 72\%\)) extends at a distance of 4.5D ahead of the cone shoulder and 2.5D behind the cone shoulder, where D is the cone diameter; (3) when the top layer has a thickness of 10D, the layer is thick enough to generate fully mobilized resistance, which can be used to interpret the shear strength profile based on conversional methods derived for single layer soil. However, due to the limited dataset, where only three sets of CPTs were conducted, the study did not propose guidelines for interpreting the CPT data.

3.1.3 Objectives

This paper aims to study the miniature CPT penetration in sand over uniform clay soils in centrifuge testing to provide guidelines in data interpretation. Figure 3-1 shows a sketch of the cone in the sand over clay soil. The LDFE analysis was first verified using the centrifuge test data. Then, a series of parametric studies were conducted to study the characteristics of cone resistance profiles to establish a soil strength interpretation framework. The soil flow mechanism was revealed to provide insights into the cone-soil interaction, especially when the cone passed through the layer interface.
3.2 METHODOLOGY

3.2.1 RITSS method

In this study, an LDFE analysis was performed by the remeshing and interpolation techniques with the small strain (RITSS) method (Hu and Randolph, 1998). The RITSS method belongs to the arbitrary Lagrangian-Eulerian (ALE) FE methods (Ponthot and Belytschko, 1998). A series of small strain analysis increments are combined with the automatic remeshing of the entire domain, followed by the interpolation of the field variables (such as the stress and soil density) from the old mesh to the newly re-established mesh. The process is repeated until the soil deforms to the expected extent (e.g., the probe penetrates to the required depth for the CPT problem). The displacement increment size and the number of steps in the small strain analysis between the remeshing stages are chosen such that the cumulative penetration between the remeshing stages remains in the small strain range and is less than half of the minimum element size, as suggested by Hu and Randolph (1998). In the study, the displacement increment in clay was chosen to be a constant value, matching the above requirement. The increment in sand could be further reduced along the LDFE process to assist convergence.

The RITSS method has been implemented into the FE package AFENA (Carter and Balaam, 1995), along with the $H$-adaptive mesh refinement cycles to optimize the mesh and minimize discretization errors in the stress concentration zone.

3.2.2 Soil models

An extended Mohr-Coulomb model is deployed to capture the stress-dependent behaviour of sand in this study. The model is developed under the critical state framework, with the goal of striking a balance between accuracy and simplicity in the LDFE analysis. It is essential to include minimum control variables to maintain stability during the analysis and encourage efficient periodical remeshing. A brief description of the CSMC model is provided here, and more details can be found in the work by Li et al. (2013).
Critical state line (CSL): A power relation is recommended by Li et al. (1999) to represent the CSL, rather than the conventional log-linear equation. The power relation shows a lower slope of the CSL at the lower stress level than that at the higher stress level, and the CSL has the potential to accurately predict the sand behaviour at low stress conditions, which has been difficult to achieve with the standard MC model.

$$e_c = e_r - \lambda \left( \frac{p'}{p_a} \right)^\xi$$  \hspace{1cm} \text{Equation 3-2}

where $e_c$ is the critical void ratio at the mean effective stress $p'$; $e_r$ is the critical void ratio at the mean effective stress diminishing to zero; $p_a$ is the reference pressure taken as the atmospheric pressure ($p_a = 101$ kPa); and $\lambda$ is the slope of the CSL in $e$ to $p'$ space.

State parameter ($\Psi$): Following the proposal of Been and Jefferies (1985), the state parameter is used to represent the soil relative position in $e$-$p$ space.

$$\Psi = e - e_c$$  \hspace{1cm} \text{Equation 3-3}

The state parameter is the governing variable since it links the soil void ratio with the soil strength parameters by the following formulas.

Dilation angle ($\psi$) and mobilized friction angle ($\phi$): By examining the best fit to sets of experimental data, a three-parameter relationship was adopted to estimate the dilation angle by the state parameter ($\Psi$).

$$\tan \psi = A \left( 1 - e^{\text{sign}(\Psi) m \Psi f} \right)$$  \hspace{1cm} \text{Equation 3-4}

where $A$ is a scaling factor, $n$ is a factor controlling the curve shape and $m$ is a factor that mainly influences the curve shape in the zone with a positive state parameter. The slope of the curve at $\Psi = 0$ is steeper with lower value of $n$ and with higher value of $m$. A steep slope of the curve (i.e. higher $d\psi/d\Psi$) can cause difficulty in numerical convergence. Thus, the model parameters $m$ and $n$ need to be adjusted to represent the sand behaviour and maintain
the numerical stability at the same time by increasing $n$ and reducing $m$ appropriately. Figure 3-2 displays the results of Equation 3-4 for Toyoura sand calibrated in this study, and its parameters are listed in Table 3-1. The energy relevant equation proposed by Taylor (1948) is adopted to link the dilation angle and the mobilized friction angle ($\phi$).

$$\tan \phi = \tan \phi_c + \tan \psi$$

Equation 3-5

where $\phi_c$ is the critical state friction angle. Combining Equations 3-4 and 5, as discussed in (2), the soil dilation and shear strength are both linked with the soil state parameter $\Psi$.

(4) Young’s modulus ($E$): The sand stiffness varies with its void ratio and stress level by

$$E = E_0 \left( \frac{2.97 - e}{1 + e} \right)^2 \sqrt{\frac{p'}{p_s}}$$

Equation 3-6

where $E_0$ is a reference stiffness and is recommended to be 6–10 MPa. Figure 3-3 displays the Young’s modulus of the sand varying with the void ratio ($e$) and the mean effective stress ($p'$) at $I_D = 60\%$ and $E_0 = 10$ MPa. The bulk and shear modulus, $K$ and $G$, can be calculated by the usual elastic relations from the Poisson ratio ($\nu$) and Young’s modulus ($E$).

(5) Yield function ($f$): The model is based on non-associated plasticity. Yield function of a classic Mohr-Coulomb model is given in Equation 3-7.

$$f(\sigma, \phi) = \sigma_1 (1 + \sin \phi) + \sigma_3 (\sin \phi - 1)$$

Equation 3-7

where $\sigma_1$ and $\sigma_3$ are major and minor principle stresses. In the model, the hyperbolic Mohr-Coulomb formulation was used, to make the yield function and plastic potentials continuous and differentiable. The detailed implementation of this formulation is described by Abbo and Sloan (1995).

The clay was modelled as a linear elastic-perfectly plastic material obeying the Tresca criterion. The Young’s modulus ($E$) and Poisson’s ratio ($\nu$) are the two elastic parameters of the model. The plastic parameter in the model is the undrained shear strength ($s_u$), which is used to determine the
size of the yield surface. The elastic parameters are independent of the soil stress state, and a constant stiffness ratio \( E/s_u = 500 \) is used in this study.

For the clay, the Poisson’s ratio is selected to be \( \nu_c = 0.49 \) for an undrained condition (close enough to 0.5 for undrained analysis, but below 0.5 to maintain numerical stability) due to its low permeability. For the sand layer, the Poisson’s ratio is taken as \( \nu_s = 0.3 \) for a fully drained condition due to its high permeability. As the miniature cone used in the centrifuge is made of either stainless steel or aluminium with a polished surface, a fully smooth cone was simulated.

### 3.3 NUMERICAL MODELLING AND VERIFICATION

#### 3.3.1 Model setup

The problem was simplified as an axisymmetric problem. The mesh setup is shown in Figure 3-4. The soil domain includes a radius of 100D and a depth of 100D to eliminate the boundary effects. For numerical simplicity, the cone tip is initially embedded in the soil with the cone shoulder level with the soil surface. Both the vertical sides adopt roller conditions to constrain the horizontal movements of the soil. The bottom boundary is in a hinge condition to constrain soil movement in both the vertical and horizontal directions.

Six-node triangular elements with three Gauss points are used. A refined mesh zone is defined around the penetrating probe in the sand layer to maintain the accuracy and stability of the analysis. When the cone penetrates the underlying clay, the refined zone in the sand layer is released, where the refined zone only applies to the area around the cone tip and the shaft up to the layer interface. In this way, the total number of elements can be substantially reduced, minimizing numerical cost and analysis time.

The soil-cone shaft and soil-cone tip interfaces were modelled as fully smooth, using nodal joint elements (Herrmann 1978).
3.3.2 Validation of the LDFE/RITSS analysis

Three sets of miniature CPTs conducted in a centrifuge by Teh et al. (2010) are used to verify the numerical model. The top sand layer and the underlying NC clay layer are formed in the Toyoura sand and Malaysia kaolin clay, respectively. Table 3-1 summarizes the parameters of the CSMC model for Toyoura sand (following Teh et al, 2010). The virgin void ratio is calculated as $e_r = 0.85 \times e_{\text{max}} + 0.15 \times e_{\text{min}}$, as suggested by Li et al. (2013). The values of $A$, $m$ and $n$ are obtained through a back analysis of the triaxial test data from Teh et al. (2010).

The setups of the three verification cases are summarized in Table 3-2. All the tests were conducted at 100 g. Prototype dimensions are obtained by multiplying the model dimension by the g-level of 100. The diameter of the model miniature cone used during the test is 10 mm; thus, the prototype diameter is $D = 1$ m. The prototype dimensions are used in the numerical analysis.

Figure 3-5 displays the comparison between the LDFE/RITSS results and the centrifuge test data. The numerical results agree very well with the centrifuge test data when the sand layers are very dense ($I_D = 88$ and 95%, Table 3-2). The LDFE analysis can accurately capture the peak resistance in the top sand layer for Test 1 and Test 2 (see Table 3-2). However, for Test 3 (i.e., $I_D = 95\%$ and $H/D = 10$), the LDFE result shows a slight underestimation but with a difference of less than 10%. Therefore, the numerical model performs reasonably well for cone penetration in sand over clay soils in centrifuge tests.

3.4 LDFE RESULTS AND DISCUSSION

The miniature cone penetration test in sand over uniform clay in the centrifuge test is investigated through a series of parametric studies. The cone diameter in the prototype ($D$), top sand layer thickness ($H$), sand relative density ($I_D$) and bottom clay layer shear strength ($s_u$) are varied in the ranges that are commonly found in centrifuge tests. All dimensions of the prototype are listed in Table 3-3.
3.4.1  Six cone penetration stages

A typical cone resistance profile in a dense sand over uniform clay is plotted in Figure 3-6 for D = 1 m, H/D = 5, I_D = 70% and s_u = 10 kPa. The resistance is the net-tip resistance, q_n (q_n = q_t – σ_v'), where q_t is the total resistance and σ_v' is the overburden pressure). There are six stages indicated on the resistance profile: Stage A_1 (d/D = 0.07) – when the cone tip is fully embedded in the soil and the cone shoulder is level with the soil surface; State A_2 (d/D = 1.5) – when the cone resistance increases fairly linearly with the penetration depth; State A_3 (d/D = 2.7) – when the cone resistance reaches its peak in the top sand layer; State A_4 (d/D = 4) – when cone resistance decreases fairly linearly as it approaches the clay layer; State A_5 (d/D = 4.8) – when the cone passes through the sand-clay interface; State A_6 (d/D = 6) – when the cone is fully embedded in the bottom layer and a steady resistance is reached in clay. The soil flow mechanisms under these six stages are discussed below.

3.4.1.1  Soil flow mechanisms

Figure 3-7 displays the soil flow mechanisms around the cone face at the six stages. At Stage A_1, the cone behaves like a surface foundation, where the surface soil near the cone shoulder heaves up and only a small portion of the sand underneath the tip moves downwards. At Stage A_2, the soil near the cone face shows a cavity expansion type of flow mechanism, and the soil besides the cone shaft moves vertically upwards within the band with a radius of approximately 0.4D. Hence, the soil surface keeps heaving, and the cone resistance increases with depth. At Stage A_3, where the cone peak resistance is reached, the soil displays mostly downward movement, and there is no more upward flow beside the cone shaft. Thus, the soil surface heave ceases. The change in the soil flow mechanism suggests that the cone senses the bottom soft clay as the soil flow is being attracted. Therefore, the cone resistance decreases after this point due to the influence of the soft clay.

At Stage A_4, the increasing influence of the bottom layer can be clearly seen as the layer interface sags downwards. Double shear planes (i.e., SP_1 and SP_2) appear in the sand layer between the cone
face and the layer interface. Above SP\(_1\), the soil moves upwards; between SP\(_1\) and SP\(_2\), the soil moves outwards and perpendicular to the cone face; and below SP\(_2\), the soil moves fairly vertically downwards towards the clay. This phenomenon could be due to the bending of the layer interface. As the interface bending encourages the upward movement of the layer interface away from the cone tip, the reappearance of the upward soil movement is observed besides the cone shaft. At Stage A\(_5\), the cone shoulder reaches the initial location of the layer interface. Due to the sagging of the soil interface, the cone tip just enters the bottom soft layer, and the cone shoulder is still above the deformed layer interface. At this stage, SP\(_2\) disappears, and SP\(_1\) remains. A similar phenomenon (e.g., two shear planes reduce to one) has been observed for a spudcan penetrating from sand to clay by both numerical studies (Qiu and Grabe, 2012) and centrifuge tests (Teh et al., 2008). However, no shear plane was observed in clay when cone penetrates from stiff layer to soft layer, as presented in Chapter 2. At Stage A\(_6\), the cone is fully embedded in the bottom soft clay. As the cone shoulder is just past the sagged layer interface, the upwards movement of the layer interface away from the cone induces upwards soil movement in the top sand beside the cone shaft and above the layer interface. The sagging and heaving of the layer interface can be explained by the bottom clay behaving as an undrained condition without volume change; thus, any sagging of the interface underneath the cone should inevitably induce heaving of the clay away from the cone. There is no sand being trapped underneath the cone when it penetrates into the bottom clay layer because the cone is smooth.

### 3.4.1.2 Soil stress development

Figure 3-8 displays the mean stress development for Stages A\(_1\) to A\(_6\). In the early stages (Stages A\(_1\) and A\(_2\)), the stress bubble grows in both magnitude and size with the penetration depth, and the stress bubble is fully contained in the sand layer. At Stage A\(_3\), the stress bubble boundary reaches the bottom soft clay. This indicates that the cone reaches the soft clay and that the cone resistance starts to decrease with further penetration, resulting in peak resistance. At Stages A\(_4\) to A\(_5\), the stress bubbles are extended into the soft clay layer, and the stresses in the sand decrease
dramatically. The stresses in the sand layer are largely released when the cone enters the soft clay layer at Stage A6; thus, the influence of the sand layer on the cone resistance diminishes, and the cone resistance is only dependent on the clay strength.

As soils fail in shear, the deviator stresses can directly display the soil failure mechanisms. Figure 3-9 shows the deviator stress (q) contours for all six stages. In addition to their similarity to the mean effective stress development, the deviator stress contours clearly show shear band formation from the cone face to the sand-clay interface, especially in Stages A4 and A5. More interestingly, in Stage A5, there is a clear shearing plane along the sand-clay interface, which is induced by the lateral spreading of the clay layer (i.e., cavity expansion). The interface shearing remains (i.e., Stage A6) until the cone passes through. When the cone enters the bottom soft clay in Stage A6, the magnitude of the deviator stress around the cone drops dramatically, as the maximum deviator stress of the clay is expected to be $q_{\text{max}} = 2 \times s_u = 20 \text{ kPa}$.

3.4.2 Parametric analysis

3.4.2.1 Effect of normalized sand layer thickness – H/D

Figure 3-10 plots the cone penetration resistance profiles with different top sand layer thicknesses at H/D = 3 to 10 (Group I in Table 3-3, $I_D = 70\%$, $s_u = 10 \text{ kPa}$, and $D = 1 \text{ m}$). All the resistance profiles show a peak resistance in the top sand layer and the same stable resistance in the bottom clay. The peak resistance, $q_p$, and its location, $H_p/D$ (as defined in Figure 3-6), increase with increasing normalized sand layer thickness. At a shallow penetration, the cone resistance profiles converge together. The resistance profiles start to deviate from the profile with a thicker sand layer (e.g., H/D = 10) when the cone reaches the soft bottom clay. The peak resistance is reached quickly after deviation, followed by a dramatic reduction in the resistance until it reaches a stable state. As the cone ‘cleanly’ penetrates into the clay layer (i.e., no sand is trapped underneath the cone), all the stable resistances in the bottom clay layer converge together, which is the ultimate cone resistance in single uniform clay with deep penetration.
3.4.2.2  Effect of relative density of sand - $I_D$

Figure 3-11 plots the cone penetration resistance profiles when the relative density of the sand varies from $I_D = 30\%$ to $90\%$ (Group II in Table 3-3, $H/D = 5$, $s_u = 10$ kPa, and $D = 1$ m). The location of the peak resistance, $H_p/D$, is mostly unaffected by the relative density of the sand (i.e. at $d/D = 2.7$ for $H/D = 5$). The peak resistance increases with the relative density, but the rate of increase increment decreases. This can be explained by the CSMC model shown in Figure 3-2. For dense sands, the state parameter ($\Psi$) is located towards the negative side (i.e. $\Psi < -0.1$), and the change in $\Psi$ of the two dense sands only induces a small change in $\tan(\psi)$. However, when the sand density becomes looser (i.e., $\Psi > -0.1$), the same change in $\Psi$ can induce a larger change in $\tan(\psi)$. The larger change in the dilation angle leads to a larger change in the friction angle (by Equation 3-4), producing a more noticeable change in the peak cone resistance.

3.4.2.3  Effect of undrained shear strength in clay - $s_u$

Figure 3-12 depicts the cone resistance profiles with various undrained shear strengths of the bottom clay from $s_u = 5$ to 20 kPa (Group III in Table 3-3, $H/D = 5$, $I_D = 50\%$, and $D = 1$ m). During the shallow penetration cone test ($d/D < 2$), all the cone resistance profiles merge. The resistance profiles start to diverge at $d/D > 2$. The peak cone resistance ($q_p$) increases with the clay undrained shear strength as more support is given to the sand layer. At the same time, the peak resistance depth ($H_p/D$) increases as well. Therefore, a stronger bottom clay increases the peak cone resistance and moves the peak resistance location towards itself.

3.4.2.4  Effect of cone diameter - $D$

Figure 3-13 shows the effect of cone diameter ($D$) on the resistance profiles as $D = 0.2$ to $2$ m (Group IV in Table 3-3, $H/D = 5$, $I_D = 70\%$, and $s_u = 5$ kPa). The cone resistance is dependent on $D$ in the sand but not in the clay. The peak resistance ($q_p$) increases with increasing $D$, and its location becomes shallower. Figure 3-14 displays the soil displacement and deviator stress contours of a small cone ($D = 0.2$ m) and a large cone ($D = 2$ m) at the same normalized depth of $d/D = 2.5$. 

3-15
When the cone with \( D = 0.2 \text{ m} \) shows upward soil movement and the deviator stress bubble is only confined near the cone area. The cone of \( D = 2 \text{ m} \) already exhibits a soil response that is the same as that described for the post peak in Figure 3-7, and the deviator stress bubble reaches the clay layer.

### 3.4.3 Peak resistance \((q_p)\) and peak distance \((H_p)\) in sand

The above discussion demonstrates that the cone peak resistance in the sand layer increases with (i) increasing thickness of the top sand layer (i.e. \( H/D \) in Figure 3-10); (ii) increasing relative density of the sand (i.d. \( I_D \) in Figure 3-11); (iii) increasing undrained shear strength of the bottom clay (i.e., \( I_D \) in Figure 3-12); and (iv) increasing cone diameter (i.e., \( D \) in Figure 3-13). By considering all these influencing parameters, the cone peak resistance can be fitted by Equation 3-8.

\[
q_p = B_0 I_D b_1 \sigma_{p v}^{b_2} s_a^{b_3}
\]

Equation 3-8

where \( \sigma_{p v} \) is the vertical effective stress at the peak resistance location, \( B_0 = 7.98, b_1 = 0.41, b_3 = 0.078 \) are constants and \( b_2 \) is a function of \( H/D \), as shown in Figure 3-15 (a). The predicted cone peak resistances from Equation 3-8 are compared with the measured cone peak resistances, and Figure 3-15 (b) shows a substantial correlation with \( R^2 = 0.99 \).

The peak resistance depth \((H_p/D)\) as shown in Figure 3-10, Figure 3-12 and Figure 3-13) in the resistance profile is a function of the in situ stress and clay undrained shear strength but irrelevant to the relative density (see Figure 3-11). The irrelevance implies a lean relationship, which can be used to find the top layer thickness \((H)\). Figure 3-16 (a) shows the relationships between \( H_p/D \) and \( H/D \) for various undrained shear strengths and for various relative densities (ranging from 30\%, 50\%, 70\% and 90\%). The top layer thickness is a linear function of the peak resistance depth for various undrained shear strengths.
Normalizing $H_p/D$ by $(s_u/\sigma_m)^\alpha$ would scale the peak depth $(H_p/D)$ for various values of $s_u$ into a linear function of only $H/D$, where $\sigma_m$ is the in situ vertical stress at the middle point of the top sand layer. Figure 3-16 (b) demonstrates this linear function; it has a high $R^2$, implying a strong relevance.

$$\frac{H_p}{D} \left( \frac{s_u}{\sigma_m} \right)^{0.11} = 0.71 \frac{H}{D} - 0.53$$

Equation 3-9

### 3.4.4 Distance $H_k$ in clay

As shown in Figure 3-6, $H_k$ is the distance measured from the initial layer interface to the kink point. Figure 3-17 shows the effects of the thickness of the sand layer ($H/D$), relative sand density ($I_D$) and clay strength ($s_u$) on the normalized kink distance ($H_k/D$). In the figure, $H_k/D$ increases with increasing $H/D$ and $I_D$, but it decreases with increasing $s_u$.

In general, the kink distance is influenced by the top layer soil that is trapped underneath the cone and the layer interface deformation when the cone is passing through. As the cone is fully smooth, no sand is trapped above the clay. Thus, the layer interface deformation becomes the determinative factor for $H_k/D$. The deformation of the layer interface is governed by the stiffness ratio between the upper and lower layers ($E_s/E_c$); when $E_s/E_c$ is much higher than 1.0, the top layer is much stiffer than the bottom layer, and the layer interface will bend into the bottom soft layer to induce large interface deformation (as shown at Stage A in Figure 3-7). As shown in Equation 3-6, the Young’s modulus of sand ($E_s$) is a function of the void ratio ($e$) and the mobilized mean effective stress ($p'$).

The void ratio is low and a high stress is easily developed during loading in a dense sand; hence, $E_s$ will be higher near the layer interface when the cone is approaching it. As the clay stiffness is defined as $E_c = 500s_u$, the clay stiffness is constant under a constant $s_u$. Thus, in Figure 3-11, it can be seen that, with the same bottom clay layer ($s_u = 10$ kPa, hence $E_c = 5,000$ kPa), $H_k/D$ increases with increasing $I_D$ (i.e., increasing $E_s/E_c$).
To express the normalized kink distance ($H_k/D$) as a function of the stiffness ratio ($E_s/E_c$), the stiffness of sand ($E_s$) is calculated using Equation 3-6 by substituting the mobilized peak resistance as $p'$ (i.e., $p' = q_p$). The stiffness of clay is $E_c = 500s_u$. An equation can be fitted for the kink distance based on the numerical results.

$$\frac{H_k}{D} = 0.22 \ln \left( \frac{E_s^\alpha}{E_c^\beta} \right) - 1$$  \hspace{1cm} \text{Equation 3-10}

where the curve fitting parameters are $\alpha = 2.2$ and $\beta = 0.93$ with $R^2 = 0.9622$. Figure 3-18 illustrates this equation. As there are fewer data on $E_s^\alpha/E_c^\beta \geq 20000$, it is suggested that Equation 3-10 needs to be used with cautious when $E_s^\alpha/E_c^\beta \geq 20000$.

### 3.5 INTERPRETATION FRAMEWORK

In this section, the previously established formulas are rearranged for data interpretation purposes. A framework is established to estimate the soil strength parameters and identify the layer interface (or top layer thickness). The bottom clay layer strength, $s_u$, needs to be interpreted first, and then the layer interface location, $H$, can be interpreted based on $s_u$. After both $s_u$ and $H$ are known, the sand layer relative density can be interpreted.

#### 3.5.1 Interpretation of the undrained shear strength of clay

As shown in Figure 3-10, the cone resistance reaches the ultimate resistance of the clay below the kink point in the profiles. The cone merely senses the impact from the above sand once it is fully embedded into the bottom clay. Therefore, the conventional interpretation method can apply to the bottom clay. The undrained shear strength, $s_u$, is derived from the net tip resistance, $q_n$, where $N_b$ is the cone bearing capacity factor. The factor can be calculated by Equation 3-11 (Ma et al., 2016).

$$N_b = 3.47 + 1.56 \ln I_r + 1.3\alpha$$  \hspace{1cm} \text{Equation 3-11}
where $I_r$ is the rigidity index and $\alpha$ is the cone roughness. In this project, the rigidity index for clay is 167 as $E/s_u = 500$, Cone roughness is assumed to be 0 for a fully smooth cone. The value of $N_b$ is calculated to be 11.5 from the equation.

### 3.5.2 Interpretation of the layer interface

The value of $H_p$ is easily identified in the resistance profile and, the soil unit weight and cone diameter are usually known. The regression equation (Equation 3-9) only has one unknown variable $H$. Therefore, Equation 3-9 is used to identify the sand layer thickness ($H$).

As $H$ occurs in two terms in the equation ($H/D$ and $\sigma_m' = H/2\gamma_0$), the method of trial and error is required to identify its value. In this study, the initial trail value is recommended as two times $H_p$.

### 3.5.3 Interpretation of the relative density of sand

As shown by the measured peak resistance ($q_p$) in Equation 3-8, the relative density of the sand, $I_D$, is a function of the cone peak resistance of the sand ($q_p$), the in situ vertical effective stress at the peak location $\sigma_m'$ and the undrained shear strength of the bottom clay ($s_u$). Equation 3-8 can be rearranged into Equation 3-12 to estimate the relative density from the measured results.

$$I_D(\%) = \frac{1}{D_0} \left( \frac{q_p}{\sigma_m' s_u} \right)^{d_1}$$

where $D_0=155.68$, $d_1=2.43$, $d_3=0.078$ and $d_2$ is a function of the top layer thickness ($H/D$). $d_2$ is identical to $b_2$ in Equation 3-8, as given in Figure 3-15 (a).

### 3.5.4 Evaluation of the interpretation formulas against existing methods

To evaluate the performance of the proposed formula for estimating the relative density of the sand, the predicted $I_D$ and the input $I_D$ used in this study are compared in Figure 3-19. As a comparison, the interpretation performed with a conventional formula derived for a single layer sand is also
shown. For Toyoura sand, the conventional formula proposed by Tatsuoka et al. (1990) is as follows:

\[ I_D(\%) = -85 + 76 \log_{10} \left( \frac{q_t - \sigma_{v0}}{\sigma_v'} \right) \]  

Equation 3-13

where \( \sigma_{v0} \) and \( \sigma_v' \) are the total overburden pressure and effective overburden pressure, respectively.

The conventional method (Equation 3-13) detects the relative density of loose sand more accurately than dense sand. The difference is due to the size difference in the sensing zone ("influence zone" in some other works) in soft or stiff soils. The size of the sensing zone increases with the soil stiffness (Lunne et al., 2002). Therefore, the sensing zone is smaller in loose sand (soft soil), and the cone resistance senses less impact from the bottom soft clay. Therefore, the fully mobilized resistance can be easily obtained in the loose sand, which is based on the interpretation of the relative density from the conventional formula. As the conventional equation underestimates the relative density in medium dense or dense sands, the necessity of developing new guidelines is confirmed. Equation 3-12 clusters the various relative densities well. However, the formula comes with limitations, where it could slightly overestimate the relative density for loose and medium dense sands (\( I_D = 50\% \) and \( 70\% \)) while underestimating the relative density for very dense sand (\( I_D = 90\% \)), but the difference (overestimation or underestimation) is within 10%.

### 3.6 CONCLUSION

In this study, the continuous penetration of a cone into sand over clay soils was simulated by LDFE analysis. The sand layer thickness, sand relative density and clay undrained shear strength were varied to study their effects on the cone resistance profiles.

For a typical resistance profile, the cone penetration could experience six stages. Stage A_1 – shallow penetration in sand with global flow mechanisms; Stage A_2 – deep penetration in sand without the influence of the bottom clay, exhibiting a localized flow mechanism; Stage A_3 – the cone reaches
the bottom clay and the peak resistance is reached; Stage $A_4$ – double shear planes are formed in the sand between the cone face and the clay surface due to the layer interface deformation; Stage $A_5$ – the cone tip enters the clay, the lower shear plane disappears and the top shear plane remains; Stage $A_6$ – the cone is fully embedded in the clay, and the cone resistance reaches its ultimate value for the clay.

The cone peak resistance, the peak location and the kink distance (the distance measured from the kink to initial sand-clay interface in the resistance profile) are affected by the sand layer thickness, cone size, sand relative density and undrained shear strength of clay. These effects have been formulated to form the interpretation framework. The interpretation of the undrained shear strength can follow conventional methods because the stable resistance in clay is merely influenced by the sand layer. The sand layer thickness can be estimated by measuring the peak resistance depth. The proposed formula for the sand relative density was evaluated against the current data. Based on the comparison of the existing interpretation formula derived for single-layer sand and the proposed formula, the proposed formula provides better predictions. However, as the formulas are proposed based on the planned ranges of soil layer profiles and soil parameters studied, any interpolation outside of the ranges needs to be conducted with caution.

Toyoura sand was modelled in this study as it has been commonly used in centrifuge tests. When different sands are encountered, the coefficients in the proposed formulas may vary. However, the proposed framework is still valuable when the coefficients can be calibrated with new data.

3.7 REFERENCE


Xu, X. (2007). *Investigation of the end bearing performance of displacement piles in sand*. University of Western Australia Perth, WA, Australia,


Figure 3-1. Schematic diagram of the CPT in sand over clay soils in the centrifuge test

Figure 3-2. Dilation angle variation with the state parameter
Figure 3-3. Young’s modulus variation with the mean effective stress for $I_D = 30\%$, $50\%$ and $90\%$.

Figure 3-4. Mesh setup with the illustrated refined zone.
Figure 3-5. Validation of the LDFE/RITSS analysis against the centrifuge data (Teh et al. 2010, in Table 3-2)

Figure 3-6. A typical cone resistance profile in sand over uniform clay (D = 1 m, H/D = 5, I_D = 70% and s_u = 10 kPa)
Figure 3-7. Soil flow mechanisms at six stages from Stage A₁ to Stage A₆ (D = 1 m, H/D = 5, I₀ = 70% and s_u = 10 kPa)
Figure 3-8. Mean stress contours for six stages from Stage $A_1$ to Stage $A_6$ ($D = 1$ m, $H/D = 5$, $I_D = 70\%$ and $s_u = 10$ kPa)
Figure 3-9. Deviator stress contours for six stages from Stage A₁ to Stage A₆ (D = 1 m, H/D = 5, I_D = 70% and s_u = 10 kPa)

Figure 3-10. Effect of the thickness of the top sand layer on the cone resistance profiles (Group I in Table 3-3, I_D = 70%, s_u = 10 kPa, and D = 1 m)
Figure 3-11. Effect of the initial sand relative density on the cone penetration resistance profile

(Group II in Table 3-3, $H/D = 5$, $s_u = 10$ kPa, and $D = 1$ m)

Figure 3-12. Effect of the undrained shear strength of the clay on the cone penetration resistance profile (Group III in Table 3-3, $H/D = 5$, $I_D = 50\%$, and $D = 1$ m)
Figure 3-13. Effect of the cone diameter on the resistance profiles (Group IV in Table 3-3, H/D = 5, I_D = 70% and s_u = 5 kPa)
Figure 3-14. Effect of the cone diameter on the soil displacement and state parameter contours at $d/D = 2.5$ with (a) $D = 0.2$ m and (b) $D = 2.0$ m (Group IV in Table 3-3, $H/D = 5$, $I_D = 70\%$ and $s_u = 5$ kPa)
Figure 3-15. (a) Coefficient \( b_2 \) in Equation 3-8; (b) Performance of Equation 3-8

Figure 3-16. (a) Relationship between \( \frac{H_p}{D} \) and \( \frac{H}{D} \); (b) Function between \( \frac{H_p}{D} \) and \( \frac{H}{D} \)
Figure 3-17. Relationships between $H_k/D$ and $I_D$ and $s_u$ ($D = 1$ m) (a) $s_u = 5$ kPa; (b) $s_u = 20$ kPa

Figure 3-18. Expression of $H_k/D$ as a function of the stiffness ratio ($E_s/\alpha/E_c/\beta$)
Figure 3-19. Interpretation results from the proposed formula (Equation 3-12) and the conventional formula (Equation 3-13)
Table 3-1. List of all the soil parameters for Toyoura sand in the CSMC model

<table>
<thead>
<tr>
<th>$e_{\text{max}}$</th>
<th>$e_{\text{min}}$</th>
<th>$e_R$</th>
<th>$\lambda$</th>
<th>$\xi$</th>
<th>A</th>
<th>m</th>
<th>n</th>
<th>$E_0$/MPa</th>
<th>$\phi/\degree$</th>
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<tbody>
<tr>
<td>0.985</td>
<td>0.611</td>
<td>0.9289</td>
<td>0.0175</td>
<td>0.75</td>
<td>0.5</td>
<td>7</td>
<td>0.75</td>
<td>10</td>
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</table>

Table 3-2. Summary of the three miniature CPT tests in the centrifuge (Teh et al. 2010)

<table>
<thead>
<tr>
<th>Test</th>
<th>Sand Thickness, H (mm)</th>
<th>Prototype sand thickness (m)</th>
<th>Sand relative density, $I_D$ (%)</th>
<th>$s_u$ at the clay surface (kPa)</th>
<th>Strength gradient, k (kPa/m)</th>
</tr>
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<tbody>
<tr>
<td>1</td>
<td>50</td>
<td>5</td>
<td>88</td>
<td>12.71</td>
<td>1.56</td>
</tr>
<tr>
<td>2</td>
<td>70</td>
<td>7</td>
<td>94</td>
<td>18.04</td>
<td>1.56</td>
</tr>
<tr>
<td>3</td>
<td>100</td>
<td>10</td>
<td>95</td>
<td>25.82</td>
<td>1.56</td>
</tr>
</tbody>
</table>

Table 3-3. Parametric study of the centrifuge CPT in sand over clay soils

<table>
<thead>
<tr>
<th>Group</th>
<th>Sand layer thickness, H (m)</th>
<th>Sand relative density, $I_D$ (%)</th>
<th>Clay undrained shear strength, $s_u$ (kPa)</th>
<th>Cone diameter, D (m)</th>
<th>Normalized top layer thickness (H/D)</th>
<th>Comments</th>
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</thead>
<tbody>
<tr>
<td>I</td>
<td>3, 5, 7, 10</td>
<td>70</td>
<td>10</td>
<td>1</td>
<td>3, 5, 7, 10</td>
<td>Explore the top layer effects</td>
</tr>
<tr>
<td>II</td>
<td>5</td>
<td>30, 50, 70, 90</td>
<td>10</td>
<td>1</td>
<td>5</td>
<td>Explore the relative density effects</td>
</tr>
<tr>
<td>III</td>
<td>5</td>
<td>70</td>
<td>5, 10, 20</td>
<td>1</td>
<td>5</td>
<td>Explore the effects of the underlying clay layer</td>
</tr>
<tr>
<td>IV</td>
<td>1</td>
<td>70</td>
<td>10</td>
<td>0.2</td>
<td>0.5</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td></td>
<td></td>
<td></td>
<td>0.5</td>
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</tr>
<tr>
<td></td>
<td>5</td>
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<td>1</td>
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</tr>
<tr>
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<td>7.5</td>
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<td></td>
<td></td>
<td>1.5</td>
<td></td>
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<td>30, 50, 70, 90</td>
<td>5, 10, 20</td>
<td>1</td>
<td>3, 5, 7, 10</td>
<td>Extensive parametric study for framework establishment</td>
</tr>
</tbody>
</table>
4 CONE PENETRATION TEST IN A THIN SAND LAYER EMBEDDED IN A UNIFORM CLAY- LDFE ANALYSIS

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ABSTRACT

The Cone Penetration Test (CPT) has been the most commonly used site investigation method in both onshore and offshore areas for many decades. Due to its widespread usage, the method attracts many researchers’ attention applying various techniques (e.g. analytical solution, numerical solution and physical tests) to study the problem. However, most of the solutions are for a cone in a single layer of soils. Recently, due to the movement of hydrocarbon exploration and production from shallow-water to deep-water area with more complex soil conditions, it has required researchers to work on site characterization of layered soils by CPT. However, there have been limited work up to date. This paper investigates CPT in three-layer soils, where a thin sand layer is embedded in a uniform clay. A parametric study using the large deformation finite element (LDFE) analysis is conducted after the LDFE method being validated against published centrifuge testing data. For the undrained clay layers, a relatively simple linear elastic-perfectly plastic material obeying a Tresca criterion is deployed to model the soil. For the drained sand layer, in order to capture the stress-dependent dilative behaviors, a critical state Mohr Coulomb model is deployed to model the sand layer. In the merit of LDFE analysis and stress-dependent modelling of sand, the insightful soil responds to the penetrating cone has been revealed for the cone penetrating from the soil surface. Based on the numerical results of cone resistance profiles, interpretation formulas are...
proposed to identify the soil interface locations, shear strength of the clay layer and relative density of the thin sand layer. The proposed formula for the relative density of the thin sand layer performs better than the existing formula, as the layered effects being well considered.

**KEYWORDS**

Cone penetration test; layered soil; critical state sand model; large deformation finite element analysis
4.1 INTRODUCTION

The Cone Penetration Test (CPT) has been widely carried out for site investigation in the past several decades. The standard cone penetrometer has a conical tip with a base area of 10 cm$^2$ (diameter = 3.57 cm) and 60° tip apex angle. As suggested by the International Reference Test Procedure (IRTP) and the American Society for Testing and Materials (ASTM) standards, it travels at a rate of 2 cm/s into the ground and records the soil resistance by the sensors installed near its cone shoulder. The standard rate is set to make the drainage condition feasible to the assumption that the fully drained and fully undrained conditions prevail for clean sand and pure clay respectively (Kim et al., 2008). Sometimes there are also sensors installed around the cone shoulder and sleeve allowing measurement of pore water pressure and sleeve friction.

Due to its continuous penetration and good repeatability, the CPT has gained broad acceptance in both onshore and offshore fields. It is also carried out in laboratory scaled model tests (e.g. centrifuge test) to detect the soil parameters of the testing sample. The success of the wide-spread application is built on the solid foundation of knowledge achieved by extensive research of a CPT in a single layer, although a rigorous theoretical analysis is extremely difficult due to the mobilized large strains by the cone. Research correlating the cone penetration resistance to soil properties commenced in the early-1960s, including limit equilibrium analysis and slip line analysis as summarized by Yu and Mitchell (1998). Various methodologies have been utilized to study the problem include analytical (e.g. limit equilibrium method by Durgunoglu and Mitchell (1973) and the slip-line method by Houlsby and Wroth (1982)), numerical (e.g. finite element analysis by Huang et al. (2004) and finite difference analysis by Ahmadi et al. (2005)) and experimental (e.g. the calibration chamber test by Arroyo et al. (2011) and the centrifuge test by Roy et al. (2019)) solutions. In these studies, both cohesive and cohesionless soils have been explored.

In contrast to the long history and much research attention for CPT in single layer soils, the two pioneering works on CPT in multi-layered soils started much later and fewer research outcomes can be found so far (Vreugdenhil et al., 1994, Van den Berg et al., 1996). A simplified solution,
based on the elastic theory for a cone in soft-stiff-soft layered soils has been proposed by Vreugdenhil et al., (1994). The results suggested a correction factor to be applied to the measured resistance in the thin stiff layer, where the layer is not thick enough to reach the fully mobilized cone resistance. Based on an Eulerian finite element approach, CPT in sand over clay and clay over sand were examined by Van den Berg et al., (1996) and the influence distance was explored with various layer stiffness ratios.

Later on, centrifuge tests have been carried out to investigate cone penetration in layered sands (Silva and Bolton, 2004, Mo et al., 2015). The transition between different layers can be observed from the cone resistance profiles. They also found that the cone could sense the impact from the soil layer ahead and behind in a zone of two cone diameters. However, no interpretation guideline was provided based on the physical tests due to the limited data.

More studies of the CPT in layered soils were conducted by cavity expansion theory and LDFE analysis recently. The cavity expansion theory was applied to study the layered sands by Mo et al., (2016). Similar to the study by centrifuge tests, no framework was provided for practical engineers to interpret the cone resistance data. A series of work has been done on the CPT in multilayered clays by LDFE analysis (Ma et al., 2016, Ma et al., 2017a, Ma et al., 2017b). Detailed mechanisms of the flow of soil during cone penetration were revealed and guidelines on interpretation of the CPT data to identify the layer interfaces and soil strength parameters were proposed.

For a CPT in a thin sand layer embedded in clay, a correction factor, determined by a function of the sand layer thickness to cone diameter, was suggested to increase the measured resistance if the sand layer is infinitely thick based on field data (Youd and Idriss, 2001). A finite difference analysis was also deployed to study the problem (Ahmadi and Robertson, 2005). It concluded with a similar suggestion for the correction factor but the stiffness ratio between sand to clay was taken into consideration as an additional variable. A simple Mohr-Coulomb model was used for the sand, and the constrained modulus and friction angle were set as stress dependent parameters. The study
proposed that the correction factor decreased with the sand layer thickness but increased with the stiffness ratio between layers. However, there are two limitations in the study: (1) the initial vertical stress is assumed to be a constant over the modelled soil body although it should increase with depth in practice; (2) the vertical downward displacement near the cone tip is assumed at 60° to the horizontal. The angle was validated by the cone travelling in a single soil layer, however, it might not be the case when the cone travels across the soil layer interface.

This paper focuses on CPT in a thin sand layer embedded in a uniform clay. LDFE analysis was performed for a cone penetrating continuously into the three-layer soils from the surface. A parametric study was conducted after the validation of the LDFE method against published physical test data. The soil flow mechanisms are revealed over the whole penetration process, including the cone passing through the layer interfaces. The undrained shear strength of the clay layers, the relative density of the thin sand layer, the thickness of the top clay and the thin sand layer were varied to study their combining effects on cone penetration resistance profiles. Based on the numerical results, the interpretation guidelines of soil layer interface location and layer characteristics are provided for use in design practice.

4.2 PROBLEM DEFINITION AND STUDY PLAN

A schematic of a standard in situ cone (D = 0.0357 m) penetrating into the clay-sand-clay soils by depth (d) is shown in Figure 4-1. Geometric variables are the top clay layer thickness (H_t) and the middle sand layer thickness (H_s). Soil strength variables are undrained shear strength (s_u) for clay and the relative density (I_D) for sand. The top clay and bottom clay are assumed with the same s_u. Table 4-1 summarizes the ranges of the four independent parameters in this study. The sand layer is embedded at a shallow depth (i.e. the top clay layer thickness is up to 40D, about 1.5 m). The strength parameters are chosen to cover the range of soft to stiff clays (i.e. s_u = 10 ~ 80 kPa) and loose to dense sands (i.e. I_D = 30 ~ 90%).
4.3 NUMERICAL METHODOLOGY AND VERIFICATION

4.3.1 Large deformation finite element analysis

Analyzing the CPT in layered soils is complex because the soil layers and layer interface deform when the cone is passing through. In the analytical study by Vreugdenhil et al., (1994), the layer deformation was simplified or ignored. Large deformation numerical analysis allows for a rigorous solution for this boundary value problem (Van den Berg et al., 1996). However, numerical analysis of the cone penetration problem is also challenging due to the large strains in the ground and around the probe (Mo et al., 2015). Therefore, the detailed stress/strain history over the penetration process and the load distribution on the probe are still not well understood.

To overcome the difficulties in the numerical analysis, a remeshing and interpolation technique with small strain (RITSS) is deployed to mitigate the mesh distortion caused by the large strains (Hu and Randolph, 1998b). The method is categorized as an arbitrary Lagrangian-Eulerian (ALE) FE method. It combines a series of small strain analysis with automatic remeshing of the whole domain and updating the coordinates in every defined interval. The field variables are interpolated from the old mesh to the re-established mesh. Another round of the small strain analysis will start on the new mesh with the interpolated field variables. In this study, the process is cycled until the desired cone penetration depth is achieved. To keep the analysis stable and accurate, the incremental displacement of each step and the number of steps between each remeshing stages were chosen to keep the cumulative deformation within each remeshing interval within the small strain range. To achieve this, the total penetration depth between each remeshing interval needed to be smaller than the half of the minimum element size (Hu and Randolph, 1998b).

The finite element based computer program AFENA (Carter and Balaam, 1995) was used in this study. The RITSS method has been implemented into the program, along with H-adaptive mesh refinement cycles to optimize the mesh and minimize discretization errors in the strain concentration area (Hu and Randolph, 1998a).
4.3.2 Constitutive model

Sand is a stress-dependent material. The response of the sand around the penetrating cone addresses the stress-dependence by (1) the decrease of the internal friction angle with increased confining stress; (2) the significant change in volume (Baligh, 1975). To capture these unique responses, the critical state Mohr-Coulomb (CSMC) model is deployed in this study for sand. As its name suggested, the model collaborates the critical state concept with the Mohr-Coulomb model, and the relationships between the soil state and the parameters in Mohr-Coulomb model are established under the critical state concept (Li et al., 2013). A full description of the model has been covered by Li et al., (2013) and Xie et al., (2020). However, a brief summary is included here as background for the subsequent discussion.

The controlling variable to link the sand state with the soil parameters is the sand state parameter ($\Psi = e - e_c$, where $e$ is void ratio at the current stress and $e_c$ is the critical state void ratio at the same stress level), as proposed by Been and Jefferies (1985). A power equation was chosen for the critical state line (CSL), to calculate the $e_c$, because it is simple and flexible, showing good accuracy for a wide range of sands. It is worthwhile to note that the power law can capture the critical state line at low stress level accurately. The CSMC model has two stress-dependent elastic factors, Young’s modulus ($E$) and shear modulus ($G$), two stress-dependent plastic factors, dilation angle ($\psi$) and friction angle ($\phi$). Table 4-2 summarizes the key equations and their coefficients required as inputs for the CSMC model. Figure 4-2 displays the equations for the CSL and the relationship between the dilation angle and state parameter. The initial void ratios and state parameters are marked in the figures. All the initial void ratios (green markers in Figure 4-2(a)) are below the CSL in the study, although the relative density varies. This is because of the low overburden pressure acting to the sand layer (low initial in-situ stress) being placed by the relatively thin top clay layer. As in Table 4-1, the maximum thickness of the top clay layer is modelled as 40D in this study, resulting
in 13.99 kPa of overburden pressure above the sand layer surface. Therefore, the results presented in this paper applies to sand layers being embedded shallowly in clay.

For clay, a relatively simple linear elastic-perfectly plastic material obeying the Tresca criterion is chosen to model the clay behaviour. The model requires two elastic parameters, Young’s modulus and Poisson’s Ratio. It only requires one plastic parameter, undrained shear strength ($s_u$) to determine the size of the yield surface. The elastic parameters are assumed as independent of the soil stress state and a constant stiffness ratio ($E/s_u = 500$) was assumed in this study.

To meet the assumption of fully drained sand and fully undrained clay, the Poisson’s ratios were set to 0.3 for sand and 0.49 for clay (close enough to 0.5 but still below to keep the analysis stable) respectively.

### 4.3.3 Mesh set-up

In this study, the cone penetration problem was simplified as an axisymmetric problem. Figure 4-3 shows the mesh with the cone penetrating into the bottom clay layer in the case where the top clay layer is 20D thick and the middle sand is 5D thick (Group IV in Table 4-1). To eliminate the boundary effects on the results, the depth of the soil domain was 20D for the bottom clay layer with the top clay and sand layer as designed in each case as listed in Table 4-1. The radius of the soil domain was the same as the total depth ($20D + H_t + H_s$). Right and left vertical sides were set as roller boundary to constrain horizontal movements with free vertical movements. The bottom boundary was set as hinge condition to constrain the soil movements in both vertical and horizontal directions. Six-noted triangular elements with three Gauss points were deployed. At the start of the analysis, the cone shoulder was embedded in the soil to avoid numerical difficulty.

To improve numerical stability and achieve high numerical accuracy, a refined zone was added to move with the cone tip, as shown in the zoom-in window in Figure 4-3. The size of the refined zone was defined as 1D from the cone central axis and 0.5D below the cone apex. The mesh was
de-refined (i.e. to make it coarse) in the area moving out of the refined zone, in order to keep the total element number low enough to maintain the numerical efficiency.

4.3.4 Verification

The AFENA program with RITSS approach has been used extensively for penetration problems in the past two decades (Hu and Randolph, 1998b, Ma et al., 2014, Wang et al., 2015). In recent years, it was used for CPT in layered clays successfully (Ma et al., 2016, Ma et al., 2017a, Ma et al., 2017b, Xie et al., 2019). The results in clays were verified against numerical finding results and physical modelling results, and both showed good agreements. For the capability of modelling sand response to cone penetration, the CSMC model has been proved capable to model pile penetrating in purely super fine silica sand by verifying against physical test data (Ma et al., 2014). Additionally, the model was used for CPT in sand over clay soils, where the sand model followed the Toyoura sand properties (Xie et al., 2020). Figure 4-4 provides further verification of modelling the CPT in layered soil composited with both clay and sand. The physical data is the mini-CPT conducted in UWA super fine (SF) silica sand over kaolin clay operated under accelerations 50 times that of Earth’s gravity in a geotechnical centrifuge (Lu, 2008). The top sand was 71 mm thick (prototype of 3.55 m) and the cone diameter was 10 mm (prototype of 0.5 m). In the numerical analysis the prototype sizes were modelled. The undrained shear strength of the underlying clay was 14.62 kPa at the sand-clay interface and increased linearly with depth at a rate of 1.24 kPa/m. The CSMC model parameters for the sand are summarized in Table 4-3 as reported by Li et al. (2013). The peak resistance in the sand layer was predicted by the numerical study to be 5040 kPa, only 2.2% lower than the measured peak resistance in the centrifuge test. The numerical model also captured the peak resistance at a similar vertical position in the profiles as in the physical model. Additionally, the trends of initial increase and the subsequent decrease due to the bottom clay were well captured.
The CSMC model was validated for a cone in purely sand as well against centrifuge test data of cone penetration in UWA supper fine silica sand (Roy et al., 2019). The tests were conducted using a mini-cone of 10 mm in diameter and under 20 g acceleration. Thus, the prototype cone diameter becomes 0.2 m. The prototype cone penetration in two sands are selected for validation: loose sand with ID = 40.95% and dense sand with ID = 72.2%. Figure 4-5 presents the LDFE numerical results against the centrifuge testing results. It can be seen that the LDFE results agree with the centrifuge test data very well for both loose and dense sands. The more oscillations of the LDFE results in dense sand is due to the numerical instability induced by higher stiffness and more softening effect of the denser sand than those of the looser sand. The largest difference between the LDFE results and the centrifuge data for dense sand is within 15%, occurred at d/D=2.

The extensive verification in the mentioned studies and the further verification case in Figure 4-4 provides confidence for the remaining LDFE results presented in this paper.

In this study, the sand and clay were modelled as Toyoura sand and Kaolin clay as their properties are well calibrated. Table 4-3 summarizes the parameters in the CSMC model for Toyoura sand as reported by Xie et al. (2020).

### 4.4 RESULTS BY PENETRATION STAGES

To explore soil response along the continuous penetration of the cone, the cone resistance profile of central case (as indicated in Table 4-1) is shown in Figure 4-6 with 10 penetration stages marked as $A_1$ to $A_{10}$. C-S represents the clay-sand interface and S-C represents the sand-clay interface. The two factors $d_{kt}$ and $d_{kb}$ are the measured distances of the kink points in the top clay layer and in the bottom clay layer to the initial interfaces (C-S and S-C) respectively. The distance of the peak resistance in sand, $d_p$, is measured from peak resistance to the original C-S interface.

There are three penetration Stages in the top clay: shallow penetration (Stage $A_1$, d/D = 2), deep penetration with stable resistance (Stage $A_2$, d/D = 15) and kink formation (Stage $A_3$, d/D = 19.35); three penetration Stages of the cone passing two layer interfaces (State $A_4$, d/D = 19.8, passing the
C-S interface, Stages A_7, d/D = 24.8 and A_8, d/D = 25, passing the S-C interface); two penetration Stages in the middle sand layer: peak resistance (Stage A_5, d/D = 21.5) and the one showing influence of the bottom clay layer (Stage A_6, d/D = 23.5); two penetration Stages in the bottom clay: kink formation (showing the diminishing influence of the sand layer, Stage A_9, d/D = 26.1) and stable resistance being reached again in clay (Stage A_{10}, d/D = 26.4). The developments of soil flow mechanisms and stresses in these Stages for the central case are discussed in detail below.

### 4.4.1 Flow mechanism

Figure 4-7 displays the displacement vectors in all penetration Stages defined in Figure 4-6. At Stage A_1, the clay is being pushed extensively vertically downwards below the cone and upwards along the shaft. The upward soil movements above the cone shoulder suggests the heaving of the soil surface, similar to the common shallow failure mode. The cone resistance keeps increasing with the increasing penetration depth under such failure model. At Stage A_2, the soil movement is directed vertically downwards and sideways to form cavity expansion type of failure mechanism, where soil surface shows no further heaving and the deep penetration flow mechanism in clay is reached. The cone resistance reaches its steady state and it is only a function only of the undrained shear strength of the current clay layer. At Stage A_3 where the cone tip touches the C-S interface, due to the stiff sand layer, there is little deformation of the interface, and the clay is squeezed horizontally between the cone face and the C-S interface. The squeezing mechanism pushes the clay above the cone shoulder moving upwards. At the same time, the movement in the sand layer is minimal. The squeezing mechanism in clay continues as the cone is passing through the C-S interface at Stage A_4. As the cone penetrates into the sand layer, the sand starts to be displaced. Similar phenomenon was observed by Wang et al. (2020) with the Particle Image Velocimetry (PIV) method when cone penetrated in soft-stiff-soft clay, as shown in plots (a) and (b) in Figure 4-8. The soil movement in stiff clay was minimal when the cone approached the stiff layer from a soft layer. Minimum deformation was observed in the soft-stiff soil interface.
Once the cone fully penetrates into the sand layer at Stage A₅, the sand is displaced extensively until cone resistance reaches its peak. After the peak in Stage A₆, since the sand starts to sense the soft clay underneath, more downwards movement of sand towards the bottom clay is displayed. The cone resistance keeps decreasing as it moves closer to the bottom clay. The dominant downwards movement in sand is not observed in Stage A₄, as the influence of the bottom clay could not reach that height in the sand layer. The flow mechanism of cone in sand was studied by Arshad et al., (2017) by Digital Image Correction (DIC) technique, their physical testing results in dense sand (Iᵩ = 85%) are presented in Figure 4-9. Due to the overburden pressure of the above clay, the flow mechanism in sand (i.e. stage A₅) revealed by this numerical study is similar to the plot (c) in Figure 4-9.

When the cone is passing the S-C interface at Stages A₇ and A₈, due to the soft bottom clay, the layer interface sags, which is very different from the cone passing the S-C interface. The cone tip touches the deformed S-C interface (i.e. sagging by 0.6D) with the soil movement being directed to the bottom soft clay. With more and more soil failure in the bottom clay, the cone resistance reduces quickly. As soon as the cone enters the bottom clay completely by detaching the deformed sand layer (i.e. S-C interface sagging by 1.1D) behind at Stages A₉ and A₁₀, the soil movement only involves clay and the above sand layer does not have much influence on the measured resistance. Hence the sharp kink point of cone resistance at Stage A₉ and stable resistance at Stage A₁₀ are observed in Figure 4-6. The PIV results, as shown by plots (c) and (d) in Figure 4-8, confirm the observed numerical results: stiff-soft interface significantly deformed by the passing cone; and minimal soil movement occurred in the stiff layer once the cone is fully in soft layer.

4.4.2 Stress development

Figure 4-10 shows the development of the mean stress from Stages A₂ to A₁₀. The mean stress around the cone face in clay is relatively low. The stress increases dramatically when the cone penetrates into the sand layer. The highest stress at the cone face is shown at Stage A₅. This explains
the peak resistance at Stage A5. The stress bubble follows the cone face during its penetration until Stage A5. The stress releases along its penetration path (i.e. along the cone shaft) when approaching the bottom clay at Stage A6. Although the stress developed in the sand layer releases when the cone passes the sand layer, the residual stress can still be observed in the sand layer along the cone shaft when the cone penetrates into the bottom clay layer (i.e. Stages A7 to A10).

The squeezing mechanism observed in Stage A3 in Figure 4-7 can be observed in Figure 4-10 as well as the stress development in top clay extending sideways in clay but with minimal stress development in the sand layer. When the cone passes through the C-S interface, the stress in sand starts to quickly develop since the cone tip touches the interface. As also shown in Figure 4-7, the C-S interface has no obvious deformation. In the contrary, when the cone approaches and passes through the S-C interface, the interface starts to deform and the stress already develops in clay before the cone touches the interface (at Stage A6). More development of stresses in clay can be observed when the cone moves closer to the bottom clay layer (Stages A7 to A10).

To demonstrate the shear plane development in the sand layer, deviator stress contours are depicted in Figure 4-11 from Stage A4 to A9. The shear plane appears after Stage A5 (i.e. the peak resistance stage). Thus, the cone resistance reduction after this stage (see Figure 4-6) can be linked to the shear plane developed in the sand layer. This development forms the pressure area at the surface of the bottom clay and triggers the deformation of the S-C interface at Stage A6. Since this stage, as the cone is approaching the bottom clay layer, the shear plane becomes shorter and the pressure area at the interface becomes smaller and the S-C interface continues to deform in Stage A7 to A8 until the cone passes the interface at Stage A9.

### 4.4.3 Stage parameter development

To further demonstrate the behaviors of sand during cone penetration, Figure 4-12 shows contours of the state parameter ($\Psi$) from Stage A4 to A9. As the state parameter of sand is $\Psi = e - e_c$, for a medium dense sand of I_D = 60% with shallow embedment (i.e. 20D, about 0.7 m below the ground
surface resulting low geostatic stress), the initial $\Psi$ is less than zero (i.e. dense sand in Figure 4-2). When the cone penetrates into the sand layer, $\Psi$ around the cone is increasing and approaching zero, where critical state line is. Figure 4-12 colors the soil at critical state in orange (i.e. $-0.02 < \Psi < +0.02$). The critical zone starts to develop when the cone enters the sand layer at Stages A4. This critical zone grows with increasing penetration of the cone. It reaches its maximum of 1.4D in width and 2.5D in height at Stage A5. The critical zone stabilizes with further penetration of the cone until the cone tip touches the deformed S-C interface. Since Stage A7, the critical zone remains its width but shrinks along the cone shaft due to stress release. The reduction of cone resistance after Stage A5 is due to the influence of the bottom clay. Once the cone enters the bottom clay at Stages A8 and A9, the critical zone reduces its size as no more sand around the cone face, and the stress in sand along the shaft keeps relaxing resulting decreased $\Psi$.

4.5 RESULTS OF PARAMETRIC STUDY

4.5.1 Effect of top layer thickness, $H_t/D$

Figure 4-13 depicts the resistance profiles for various top clay layer thickness ranging from 5D to 40D (see Group I in Table 4-1), with layer interfaces marked on the plot. Figure 4-13 (a) shows the penetration depth from the soil surface. The resistance profiles in the top and bottom clay layers are coinciding for all cases where a steady state in the clay layer is reached. The peak resistance in the sand layer increases with increasing top layer thickness. This is because the sand strength and stiffness are a function of stresses (see Equation 4-2 in Table 4-2). With increasing top layer thickness, the stress level in sand is increasing, so are the sand strength and stiffness. The high sand strength induces high peak resistance in sand.

The variation of peak resistance in sand with top layer thickness can be seen more clearly in Figure 4-13 (b). The vertical axis is set as the penetration depth below the C-S interface as $d_s = d - H_t$, as defined in Figure 4-1. Figure 4-13 (b) shows more clearly that $q_p$ increases with increasing $H_t/D$. However, the increment in the peak resistance slows down when the top layer thickness reaches
Hₜ/D = 20. This may be due to the sand strength increases faster with stress increase at lower stress level than that at higher stress level. (as suggested by Equation 4-5 and Figure 4-2(b)). Moreover, with the increasing top layer thickness, the peak resistance in sand is reached earlier with higher Hₜ/D, which is observed as smaller dₚ in the profiles. This means that, with higher Hₜ/D, the sand layer is embedded deeper and experiencing higher stresses, hence sand layer stiffness is higher. As the stiffness and strength in the top and bottom clays are constant, the stiffness ratio between sand and clay (Eₛ/Eₐ) becomes higher with higher Hₜ/D. The higher stiffness ratio of Eₛ/Eₐ could be the cause of the earlier cone peak resistance presented in deeper embedded sand layer, as the sand layer is more rigid.

The effect of top layer thickness is also observed on the kink points in the top and bottom clay layers (i.e. dₖt and dₖb in Figure 4-6), as indicated in Figure 4-13 (b). The kink distances in top and bottom clays are plotted in Figure 4-14 as a function of Hₜ/D. It is apparent that, with increasing top clay layer thickness, the kink distance in the top clay (dₖt) decreases while the kink distance in the bottom clay (dₖb) increases. This is because that, with increasing Hₜ/D, the stiffness ratio of Eₛ/Eₐ is increasing as explained above. The higher Eₛ/Eₐ induces less deformation of the C-S interface (with more squeezing mechanism of clay) and more deformation of the S-C interface (with more sagging of the interface), hence lower dₖt and higher dₖb. Both dₖt and dₖb stabilizes at dₖt/D = 0.65 and dₖb/D = 1.2 for Hₜ/D > 15 (sᵤ = 20 kPa, I_D = 60%). The measurements of dₖt and dₖb can be utilized to locate the layer interfaces later.

4.5.2 Effect of sand layer thickness, Hₛ/D

Figure 4-15 (a) shows the effect of sand layer thickness on the CPT resistance profile (Group II in Table 4-3). The sand layer thickness covers Hₛ/D = 3 & 5 and their results are compared with the infinite sand layer case (i.e. clay over sand). The resistance profiles in the top clay layer converge together with the same kink point identified as dₖt/D = 0.65 with different sand layer thicknesses. The peak resistance in the sand layer increases with increasing sand layer thickness, with no peak
observed for the clay-sand case. This is further evidence that the reduction of cone resistance in a thin sand layer after its peak is due to the influence of the bottom soft clay. Regarding the bottom kink distance, it is registered as \( d_{kb}/D = 1.2 \) for both clay-sand-clay cases.

Figure 4-15 (b) shows the cone resistance normalized by the overburden pressure (\( \sigma' \)). The normalized resistance reaches a constant after cone penetrates 4D below the initial clay-sand interface. It implies the cone resistance has been fully mobilized without impact from the adjacent layers at this stage. Hence, the cone resistance profile only reflects the condition and properties of the current soil layer, which can be interpreted by the conventional method derived for a single sand layer.

### 4.5.3 Effect of undrained shear strength of the clay, \( s_u \)

Figure 4-16 shows the effect of clay strength on the cone resistance profiles (Group III in Table 4-3). As stated before, the undrained shear strengths of the top and bottom clay layers are identical. The peak resistance in sand increases with increasing clay strength. At the same time, the location of the peak resistance, \( d_p \), increases with the clay shear strength. This can be linked to the stiffness ratio of \( E_s/E_c \). As \( E_s \) is kept unchanged and \( E_c \) is increasing with increasing \( s_u \), the stiffness ratio of \( E_s/E_c \) is decreasing with increasing \( s_u \), \( q_p \) appears later with lower \( E_s/E_c \) resulting in larger \( d_p \). This is same as the discussion above that the cone resistance can be influenced by the bottom clay earlier (i.e. smaller \( d_p \)) when the stiffness ratio of \( E_s/E_c \) is higher in Figure 4-13 (b). However, the variation of \( s_u \) does not affect the kink points in both top and bottom clays, as \( d_{kt}/D = 0.65 \) and \( d_{kb}/D = 1.2 \). This strengthens the observation in Figure 4-14 that, with \( H_t/D > 15 \), \( d_{kt}/D \) and \( d_{kb}/D \) become constant and are not influenced by \( E_s/E_c \).

### 4.5.4 Effect of relative density of the sand, \( I_D \)

Figure 4-17 shows the effect of relative density of sand on the cone resistance profiles (Group IV in Table 4-3). The peak resistance in sand increases with its increasing relative density. However, the increase in the peak resistance becomes minimal when \( I_D \geq 60 \% \). This is because, in Figure
4-2, a denser sand has a lower initial void ratio, thus lower initial state parameter (e.g. Point 1 in both Figure 4-2(a) and 4-2(b) for $I_D = 90\%$). When $I_D$ changes between 90% and 60% (i.e. Points 1 to 4 in Figure 4-2), the dilation angles are similar (Figure 4-2(b)), hence the sand friction angles and strengths are similar as suggested by Equation 4-5. Therefore, similar cone resistance profiles in sand are displayed in Figure 4-18 for $I_D = 60 \sim 90\%$. However, when the sand becomes looser as points 5 to 6 of $I_D < 60\%$, the dilation angles become more different in Figure 4-2 (b), hence the sand peak friction angle and strength. This could be the main reason for the more different resistance profiles in sand for $I_D = 30\%$ and 50%.

The kink distance in the top clay is not affected by the relative density of sand as $d_{kt}/D = 0.65$. The kink distance in the bottom clay is kept as $d_{kb}/D = 1.2$, except the very loose sand layer ($I_D = 30\%$) as $d_{kb}/D = 0.78$. This might be due to the much lower dilation angle shown in Figure 4-2 (i.e. Point 6) that induces much lower stiffness ratio of $E_s/E_c$ relative to the cases with $I_D = 50 \sim 90\%$. The lower ratio of $E_s/E_c$ can potentially reduce the sagging of the S-C interface, even when $H_t/D = 20 > 15$, hence less $d_{kb}$.

**4.6 INTERPRETATION OF CPT DATA**

**4.6.1 Identification of layer interface location**

As the cone resistance profiles plotted by the normalized penetration depth ($d/D$) are consistent, the locations of the interfaces of clay-sand and sand-clay can be identified by the kink distances of $d_{kt}$ and $d_{kb}$. Based on the observations discussed above, the kink distances in the top and bottom clays are only a function of the top layer thickness ratio of $H_t/D$, except one case when the relative density of sand is very low as $I_D = 30\%$, $I_D$ affects $d_{kb}$. Otherwise, all other factors, such as $H_s/D$, $s_u$ and $I_D$ (when $I_D > 30\%$), have minimal effect on the kink distance. Moreover, when the sand layer is embedded deeply enough (i.e. $H_t/D \geq 15$), the kink distances become constant as $d_{kt}/D = 0.65$ and $d_{kb}/D = 1.2$. When the layer is embedded in a shallow depth ($H_t/D = 5$ and 10), the kink distance is a function of $H_t/D$. 

4-17
For a relatively dense sand of \( I_D \geq 50\% \), the kink distances can be expressed as functions of the top layer thickness. Based on Figure 4-14, the kink distance in the top clay is expressed by Equation 4-6, and the kink distance in the bottom clay is expressed by Equation 4-7. Figure 4-14 shows the good fit of the equations. The layer interface can be interpreted by these Equations. Since the \( H_t \) is unknown initially, iterations are needed. It is recommended to use the constant values \( (d_{kt}/D = 0.65\) and \( d_{kb}/D = 1.2) \) to initialize the iteration.

\[
\frac{d_{kt}}{D} = \begin{cases} 
-0.01 \frac{H_t}{D} + 0.8, & \frac{H_t}{D} < 15 \\
0.65, & \frac{H_t}{D} \geq 15 
\end{cases} \quad \text{Equation 4-6}
\]

\[
\frac{d_{kb}}{D} = \begin{cases} 
0.03 \frac{H_l}{D} + 0.75, & \frac{H_l}{D} < 15 \\
1.2, & \frac{H_l}{D} \geq 15 
\end{cases} \quad \text{Equation 4-7}
\]

4.6.2 Interpretation of undrained shear strength of clay

The interpretation of the undrained shear strength in a uniform clay is straightforward. The undrained shear strength can by interpreted as

\[
s_u = \frac{q_c}{N_b} \quad \text{Equation 4-8}
\]

where \( N_b \) is the cone bearing factor. For a single layer clay, the cone bearing factor increases with depth from a value at the soil surface (Ma et al., 2016). It is associated with the common shallow failure mechanism, as shown in Stage A1 in Figure 4-7. When the cone penetrates deeply in the clay, the localized cavity expansion failure mechanism (see the Stage A2 in Figure 4-7) leads to a constant cone bearing factor. In Figure 4-13 (a), it can be noted that the cone resistance in the top clay senses minimal influence from the sand layer until the top kink point (where \( d_{kt} \) is registered). It can be confirmed by the stress distribution at Stages A2 and A3 in Figure 4-10: the mean stress is
mainly developed around the cone tip in clay until the tip touches the clay-sand interface, even at Stage A3, the stress development in the sand is minimum compared with that in clay. Therefore, the paper will not suggest new guideline to interpret the CPT data in clay layers. Existing interpretation methods for CPT data in clay layers should be applicable. One example is the formula proposed for the top clay layer in a soft-stiff-soft clays by Ma et al. (2016), as shown in Equation 4-9.

\[
N_b = \begin{cases} 
4.45 - 0.114 \left( \frac{d}{D} \right)^{1.5} + 3.31 \left( \frac{d}{D} \right)^{0.5}, & \frac{H_t - d_{at}}{D} \leq 11 \\
11.5, & \frac{H_t - d_{at}}{D} > 11 
\end{cases} 
\]

Equation 4-9

### 4.6.3 Interpretation of relative density of sand

To interpret the relative density of sand, the measured peak resistance can be utilized. At this stage, the undrained shear strength of the surrounding clay and the top clay layer thickness are assumed as known variables based on the first two steps of interpretation. As discussed in Figures 4-10, 12, 13 and 14, the peak resistance is a function of both clay relevant variables (top clay thickness and undrained shear strength) and sand relevant variables (relative density and layer thickness). The results are collected and summarized in Table 4-4. A regression formula can be fitted as Equation 4-10, with the $R^2$ of 0.813, as shown in Figure 4-18 (a),

\[
I_D = \frac{C_0 \left( \frac{d_p}{D} \right) \left( \frac{q_p}{p_s} \right)}{\left( \frac{S_u}{\gamma D} \right)^{\alpha} \left( \frac{H_t}{D} \right)^{\beta}} + C_1 
\]

Equation 4-10

where $C_0$ is 0.36, $C_1$ is -0.18, $\alpha$ is 0.85. $\beta$ is 0.34 for the top layer is thick enough ($H_t/D \geq 15$). When the top clay layer is less than 15D thick, $\beta$ increases to 0.8 and 0.5 for $H_t/D = 5$ and 10 respectively, to guarantee the highest correlation. Figure 4-18 (b) visualizes the variation of $\beta$. All the variables in the formula have been non-dimensionalized.
4.6.4 Assessment against the conventional interpretation method

As discussed in the introduction section, a correction factor is suggested by Youd and Idriss (2001), to multiply the peak resistance in the interbedded sand layer before applying the conventional method to interpret the sand layer. A conventional method is the formula derived for single layer sand, as shown in Equation 4-11, where $C_0 = 0.175$, $C_1 = 0.5$ and $C_3 = 3.1$. The correction factor is a function of the normalized sand layer thickness ($H_s/D$), as shown in Equation 4-12.

$$I_D(\%) = \frac{100}{C_2} \ln \left( \frac{q_1 (\text{MPa})}{C_0 (\sigma_\gamma)^{C_1}} \right)$$

Equation 4-11

$$K_H = 0.25[((H_s / D) / 17) - 1.77]^2 + 1.0$$

Equation 4-12

They are used to interpret the numerical results with the interpretation results by Equation 4-10, to demonstrate the advantage of the proposed formula in the study. The comparison results are depicted in Figure 4-19. The method by Youd and Idriss (2001) overestimates the relative density for loose to medium dense sand ($I_D$ up to 75%) while underestimates the very dense sand ($I_D = 90\%$). In contrary, the proposed Equation 4-10 can estimate the relative density fairly accurately over the whole range of the sand density (from 30\% to 90\%). Although some defects are still existing, the difference between the predicted relative density and the input value is within 10\%. It is believed that the non-consideration of the precise impact from the surrounding clay by Youd and Idriss (2001) results its large deviation in the interpretation results from the input value.

4.7 CONCLUSION

This paper studies the standard cone penetrating in the thin sand layer embedded in a uniform clay by the LDFE analysis. The numerical model was verified with the centrifuge testing data of a CPT in a sand over clay soil. Good agreement was achieved providing confidence in further simulating the CPT in layered soils of sand and clay. The paper examines the soil responses during its penetration through different layers, including soil flow mechanisms, stress development in all
layers and state parameter variation in sand. In the top clay, the soil shows responses identical to the cone penetrating in a single uniform clay layer, until the cone tip touches the sand layer surface, leaving a kink \( (d_{kt}) \) registered in the top clay layer. Similarly, the soil movement is restrained in bottom clay once the cone passes through the sand layer, leaving another kink \( (d_{kb}) \) registered in the profile. When the top clay layer thickness is larger than 15D (D is cone diameter), the measured distance of the kinks to the original interfaces are constants, \( d_{kt} = 0.6D \) and \( d_{kb} = 1.2D \). When the top clay layer thickness is less than 15D, the kink distances are functions of the top layer thickness ratio of \( H_t/D \) (\( H_t \) is top clay layer thickness). When the cone is passing the clay-sand layer interface, squeezing mechanism is observed in the clay layer since the cone is entering the stiff sand layer from the top soft clay layer. However, when the cone is passing the sand-clay layer interface, the sagging of the layer interface is observed since the cone is entering the soft bottom clay layer from the stiff sand layer. The squeezing mechanism in the top clay and the sagging in the bottom clay make the kink distance in the top clay lower than that in the bottom clay, which can be used to locate the layer interfaces. The cone peak resistance in the sand layer is a function of all the variables considered in the study, including the sand relative density, sand layer thickness, top clay layer thickness and undrained shear strength of the surrounding clay. The peak resistance location \( (d_p) \) is also influenced by the above factors, the influence can be summarized by the stiffness ratio between sand and clay \( (E_s/E_c) \).

A regression function of peak resistance is fitted to the dataset collected from the study, with high correlation achieved. The function is re-arranged to interpret the relative density of the sand. Relative to the existing solutions, the proposed prediction formula works much better over a wide range of sand densities. The good performance is believed due to the better consideration of the surrounding clay than the previous solutions. However, as the formulas are proposed based on the planned ranges of soil layer profiles and soil parameters, any interpolation outside of the ranges needs to be conducted with caution.
4.8 REFERENCE


Paper draft on CPT in a thin sand layer embedded in clay

Figure 4-1. Schematic plot of the cone penetration in clay-sand-clay soil

Figure 4-2. (a) Critical state line; (b) Relationship between dilation and state parameters
Figure 4-3. Mesh set-up of the cone penetrating into the bottom clay ($H_t/D = 20$, $H_s/D = 5$)

Figure 4-4. Verification of the numerical analysis against centrifuge test data from Lu (2008)
Figure 4-5. Verification of the numerical analysis against centrifuge test data from Roy et al. (2019)
Figure 4-6. A typical CPT resistance profile with 10 penetration stages ($H_t/D = 20$, $H_s/D = 5$, $I_D = 60\%$ and $s_u = 20$ kPa)
Figure 4-7. Soil flow mechanisms from Stage A₁ to A₁₀ (Hₜ/D = 20, Hₛ/D = 5, I₀ = 60 % and sᵤ = 20 kPa)
Figure 4-8. Soil flow mechanisms of cone penetration in soft-stiff-soft clay by PIV method: (a) $d/D = 6.5$; (b) $d/D = 8.3$; (c) $d/D = 14.3$; (d) $d/D = 17.1$, from Wang et al. (2020) (in the figures, $d$- vertical distance from surface of sample to a point; $D_c$- radius of cone penetrometer; $x$- horizontal distance from a point to the cone penetration axis;
Paper draft on CPT in a thin sand layer embedded in clay

\[ \sigma_s = 0 \text{kPa}, \ D_R = 85\% \]

\[ h^* = 2r_c, \]

Cone incremental displacement = 0.13r_c

Scale for incremental displacement vectors:

(a)

(b)
Figure 4-9. Soil flow mechanisms of cone penetration in sand by DIC method: (a) d/D = 2; (b) d/D = 6; (c) d/D = 22, from Arshad et al., (2017) (in the figures, $\sigma_s$- applied surcharge pressure; $D_R$- relative density of the sand; $h^*$- cone penetration depth measured from the top of the soil sample; r- horizontal distance from a point to the cone penetration axis; $r_c$- radius of cone penetrometer; z- vertical distance from surface of sample to a point.)
Figure 4-10. Contours of mean stress from Stage A₂ to A₁₀ (Hᵢ/D = 20, Hₛ/D = 5, Iₐ = 60 % and sᵤ = 20 kPa)
Figure 4-11. Contours of deviator stress from Stage A₄ to A₉ (Hᵢ/D = 20, Hₛ/D = 5, Iₑ = 60% and $s_u = 20$ kPa)
Figure 4-12. Contours of state parameter from Stage A₄ to A₉ (Hₒ/D = 20, Hₛ/D = 5, Iₒ = 60 % and sᵤ = 20 kPa)
Figure 4-13. Effect of top layer thickness on cone resistance profile ($s_u = 20$ kPa, $I_d = 60\%$) (a) with normalized penetration depth of $d/D$; (b) with normalized penetration depth in sand layer of $d_s/D$.

Figure 4-14. Effect of top layer thickness on kink distances (a) kink in the top clay, $d_{kt}$ (b) kink in the bottom clay, $d_{kb}$.
Figure 4-15. Effect of sand layer thickness on cone resistance profile ($H_s/D = 20$, $I_D = 60$ % and $s_u = 20$ kPa): (a) cone resistance profile, (b) normalized cone resistance profile.
Figure 4-16. Effect of undrained shear strength on cone resistance profile ($I_D = 60 \%$, $H_t/D = 20$ and $H_s/D = 5$)
Figure 4-17. Effect of relative density of sand on cone resistance profile ($s_u = 20$ kPa, $H_t/D = 20$ and $H_s/D = 5$)

Figure 4-18. (a) Interpretation formula for the relative density of sand; (b) Coefficient, $\beta$. 

$y = 1.0082x$  
$R^2 = 0.8127$
Figure 4-19. Comparison between the interpretation results by the proposed formula in this paper (Equation 4-10) and the method by Youd and Idriss (2001)
Table 4-1. Summary of the parametric study plan

<table>
<thead>
<tr>
<th>Group</th>
<th>Normalized top clay layer thickness (H_t/D)</th>
<th>Normalized sand layer thickness (H_s/D)</th>
<th>Clay undrained shear strength, s_u (kPa)</th>
<th>Sand relative density, I_D (%)</th>
<th>Notes</th>
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<td>I</td>
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<td>5</td>
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<td>IV</td>
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<td>5</td>
<td>20</td>
<td>30, 50, 60, 75, 90</td>
<td>Explore the effect of the sand shear strength</td>
</tr>
</tbody>
</table>

The bold font is for the central case (H_t/D = 20, H_s/D = 5, I_D = 60 % and s_u = 20 kPa)

Table 4-2. Summary of the key equations in the CSMC model

<table>
<thead>
<tr>
<th>Critical state line</th>
<th>e_c = e_r - λ \left( \frac{p'}{p_a} \right)^{\xi}</th>
<th>e_c - critical state e at the mean effective stress p'; e_r - critical state e at the mean effective stress diminishing to zero; p' - mean effective stress; p_a - atmospheric pressure</th>
</tr>
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<tbody>
<tr>
<td>Young’s Modulus (E)</td>
<td>E = E_0 \frac{(2.97 - e)^2}{1 + e} \sqrt{\frac{p'}{p_a}}</td>
<td>Equation 4-2; E_0 - reference pressure;</td>
</tr>
<tr>
<td>Shear Modulus (G)</td>
<td>G = \frac{E}{2(1 + v)}</td>
<td>Equation 4-3; v - Poisson ratio;</td>
</tr>
<tr>
<td>Dilation angle (ψ)</td>
<td>tan ψ = A \left( 1 - e^{\xi \Sigma (Ω)</td>
<td>Ψ</td>
</tr>
<tr>
<td>Friction angle (φ)</td>
<td>tan φ = tan φ_c + tan ψ</td>
<td>Equation 4-5; φ_c - critical state friction angle;</td>
</tr>
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Table 4-3. Values of the parameters in the CSMC model

<table>
<thead>
<tr>
<th>Sand</th>
<th>$e_{max}$</th>
<th>$e_{min}$</th>
<th>$e_{f}$</th>
<th>$\lambda$</th>
<th>$\xi$</th>
<th>A</th>
<th>m</th>
<th>n</th>
<th>$\phi/°$</th>
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<td>Toyoura</td>
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Table 4-4. Summary of the results in the parametric study

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<tr>
<th>No.</th>
<th>$H/D$</th>
<th>$H_{p}/D$</th>
<th>$s_{u}$/kPa</th>
<th>$I_{D}$ (%)</th>
<th>$d_{f}/D$</th>
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5 CONE PENETRATION TEST IN A THIN SAND LAYER SANDWICHED BETWEEN DIFFERENT CLAY LAYERS- LDFE ANALYSIS

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ABSTRACT

The cone penetration test (CPT) has been the most common method adopted in both onshore and offshore site investigation practices for decades. Many formulas have been proposed to interpret cone resistance profiles in single-layer soils. However, layered soil is less studied, and the layering effect is not well included in CPT data interpretation. This paper studies the CPT in a thin sand layer sandwiched between two clay layers with different undrained strengths. Large deformation finite element analysis was performed to simulate continuous cone penetration from the soil surface. An extended critical state Mohr-Coulomb model was utilized to capture the stress-dependent response for the drained sand. The Tresca failure criterion was used for the undrained clay. Based on a comprehensive parametric study, two modes of resistance profiles were identified for a stiff top clay with a soft bottom clay and a soft top clay with a stiff bottom clay. Insightful soil flow mechanisms and stress development were revealed to explain the characteristics observed in the two profiles. The cone resistance profiles were examined by the various sand layer thicknesses, relative densities and undrained shear strengths of clay. A linear function was proposed for the relationship between the peak resistance and its influential variables. Two sets of coefficients were provided for the linear function for the two profiles. Sand relative density can be
estimated through the rearrangement of the linear function, as well as through the suggested data interpretation methods provided in the paper.

**KEYWORDS**

Cone penetration test; Layered soil; Large deformation finite element analysis; Critical state sand model.
5.1 INTRODUCTION

The practice of inserting a rod into soft soil to find embedded stiff layers can be dated back to the 19th century. The first recognizable form of the cone penetration test (CPT) was developed in the Netherlands and involved the identification of the ultimate bearing capacity of piles in 1934 (Barentsen, 1936). The early form of the CPT relied on a mechanical system of pushing rods to measure the rod tip resistance. In the late 1960s, the electric penetrometer became available for general usage, enabling the continuous and automatic recording of resistance (Walker and Yu, 2010). Sensors to measure the friction force along the probe shaft and pore water pressure around the cone shoulder were later introduced onto the probe to provide additional information on the tested soil. Currently, the CPT is the most widely used in situ test method to investigate sites in geotechnical practice because it provides a continuous profile and avoids the difficulty of retrieving undisturbed soil samples for laboratory tests. Due to its reliability and repeatability, the CPT has been used to detect various soil properties and to delineate between layers of different soil types and states (ISSMGE, 1999).

The traditional design charts or formulas to identify soil properties based on CPT data were built through CPTs in uniform soils with a single layer (Silva and Bolton, 2004). The application of such an interpretation method directly to layered soils could be faulty. This is because the sensing zone of a cone can span several cone diameters both behind and ahead of the cone tip. When the sensing zone is across layer interfaces, the measured quantities (e.g., cone tip resistance, shaft friction and pore water pressure) not only represent the properties of the local soil around the cone tip but also include the impact from other layers. The size of the sensing zone (or the sphere of influence as it is called in some studies) increases with the soil stiffness. The sensing zone can be as small as two to three cone diameters in soft soils, whereas it can be up to 10 or even 20 cone diameters in stiff soils (Lunne et al., 2002). It could underestimate the soil strength of the embedded layer when it is not thick enough to mobilize the full resistance (i.e., the resistance when the embedded layer is infinitely thick).
The inability to accurately capture the thin layering effect may not be of importance in some areas of practice, but there are certain situations when the layering effect could be crucial. For example, the existence of a thin sand layer can be very important in soil liquefaction analysis (Boulanger and DeJong, 2018). Through examining the field data (Yost et al., 2019) and studying historical cases (Munter et al., 2017), thin layering effects, among other factors, could contribute to an accumulation of conservatism or bias in predicted liquefaction behaviours. Another driver for studying the thin layering effect is the substantial use of centrifuge tests to study offshore structures in layered soils. Centrifuge tests have been used extensively to study the structure and soil interaction problem in layered soil due to their ability to model the prototype stress response on the structure at laboratory scales (Ullah et al., 2017, Teh et al., 2010, Hossain and Randolph, 2010).

The CPT can be conducted in flight to detect the strength profile of the layered soil. The smallest that the centrifuge cone can be made is around 7-10 mm in diameter (effectively 0.7 – 1 m in prototype dimensions if tested at 100 g), to match the requirement from the scaling effect in centrifuge (Garnier et al., 2007). Therefore, in the test setup, the soil layers can be as small as three cone diameters. Without corrections to the cone resistance measured in a thin layer, the soil characteristics of the thin layer cannot be estimated accurately (Teh et al., 2006).

There are relatively few studies on CPTs in layered soils. The existing research has mainly focused on idealized soil profiles and two or three uniform soil layers in different sequences: weak soil over strong soil or vice versa and a strong layer embedded in soft soils or vice versa. Methodologies have included elastic analysis (Vreugdenhil et al., 1994, Yue and Yin, 1999), cavity expansion theory (Mo et al., 2016), chamber testing (Tehrani et al., 2017), centrifuge testing (Mo et al., 2015) and field data analysis (Youd and Idriss, 2001). Recently, large deformation finite element (LDFE) analysis has emerged for CPTs in layered soils under practical stress fields (Ma et al., 2016, Ma et al., 2017).
CPTs in clay-sand-clay soils have been researched through analytical analysis, field data summary and numerical studies. The analytical solutions were obtained by elastic analysis to investigate the “error” between the resistance in a thin embedded stiff layer and the ultimate resistance in an infinitely thick layer of the same stiff soil (Vreugdenhil et al., 1994). The study suggested that the “error” was a function of the middle stiff layer thickness and the stiffness ratio between adjacent layers. Based on the results, a correction factor was suggested to multiply the peak resistance in the stiff layer before using the resistance to classify the stiff soil (Robertson and Fear, 1995). Although the factor is a function of both layer thickness and stiffness ratio, a lower band (corresponding to the smallest stiffness ratio in their paper) was suggested for conservative purposes in soil liquefaction assessment. Later, an even more conservative band of the factor was suggested based on the analysis of the field data (Youd and Idriss, 2001). A numerical examination of the band solution suggests that it should only be applicable to thin, young and normally consolidated sand layers embedded in soft normally consolidated clays (Ahmadi and Robertson, 2005). Caution is needed when applying this procedure in the other cases. The numerical procedure used for the previous study has been verified against chamber test data and previous numerical results, which showed good agreement. However, the verification was conducted for uniform soil, either clay or sand, and not for layered soils.

In the above numerical studies, the stiffness properties of the top and bottom clays were assumed to be identical instead of considering an even more complex situation: the top clay different from the bottom clay. The stress history over the penetration process, especially when the cone goes across the layer interface, and the detailed soil flow was not revealed. Therefore, this study aims to address such limitations.

In this study, the CPT in sand layers sandwiched by different clays is studied by LDFE analysis, along with a stress-dependent model being deployed to capture the sand response. A parametric study is conducted to reveal the impacts of influential factors on the cone resistance profile. The
influencing factors include clay strengths in the top and bottom layers (i.e., stiff layer on top or soft layer on top), sand layer thickness (i.e., 3D to 10D to keep it thin; D is the cone diameter), and sand layer relative density (i.e., $I_D = 30 \sim 90\%$). Based on the results of this parametric study, an interpretation formula for the thin sand layer will be proposed.

5.2 METHODOLOGY

5.2.1 RITSS method

Studying the soil response to probe penetration is a large strain problem. To overcome the mesh distortion, the remeshing and interpolation technique with small strain (RITSS) method is deployed (Hu and Randolph, 1998b). The RITSS is a type of arbitrary Lagrangian-Eulerian (ALE) FE method. It starts with a series of small strain incremental displacement analyses with the initial mesh, followed by updating the coordinates of all nodes in the mesh by the displacements calculated in the first steps and automatically remeshing the whole domain based on the updated domain boundaries, which include layer boundaries (or layer interfaces) in layered soils. On the newly established mesh, the field variables (such as soil state parameters) are interpolated from the old mesh. Another round of the small strain analysis is carried out on the new mesh. The process is cycled until the desired probe penetration depth is achieved (e.g., the expected penetration depth of the cone). It should be noted that the total displacement between each remeshing cycle should remain in the small strain range and should be smaller than half of the minimum element size to maintain both the efficiency and accuracy of the simulation (Hu and Randolph, 1998b). The finite element-based computer program AFENA (Carter and Balaam, 1995) is used in this study. H-adaptive mesh refinement cycles are also implemented in the program to optimize the mesh density and minimize the discretization errors in the stress-concentrating zones (Hu and Randolph, 1998a).

5.2.2 Soil constitutive model

As the tip resistance in granular material often exceeds the stress level encountered in other geotechnical applications, a realistic soil response at an elevated stress level must be considered to
calculate the resistance accurately (Baligh, 1975). On the other hand, the soil model used for large
deformation problems cannot be too complex, as it would increase the computational cost
dramatically and cause numerical instability (i.e., non-convergence). To address both issues, the
critical state Mohr-Coulomb (CSMC) model (Li et al., 2013) is utilized in this study to capture the
stress-dependent response of sand. The model introduces the critical state concept into the classical
Mohr-Coulomb (MC) model by linking the friction angle and dilation angle in the MC model with
the soil state parameter ($\Psi$). The soil state parameter is the difference between the current soil void
ratio ($e$) and the critical state void ratio ($e_c$) at the same stress level, as given in Equation 5-1.

$$\Psi = e - e_c$$  \hspace{1cm} \text{Equation 5-1}$$

Detailed development of the model can be found in (Li et al., 2013). Here, a brief description
provides the necessary base for the discussion in the following sections. There are two elastic
parameters: Young’s modulus ($E$), shear modulus ($G$) and Poisson’s ratio ($v$). Poisson’s ratio is a
constant in this study at 0.3, while both Young’s modulus and shear modulus vary with the stress
level and void ratio. The formula to calculate Young’s modulus is given in Equation 5-2.

$$E = E_0 \frac{(2.97 - e)^3}{1 + e} \sqrt{\frac{p'}{p_a}}$$  \hspace{1cm} \text{Equation 5-2}$$

where $E_0$ is suggested as 6~10 MPa; $e$ is the void ratio at the mean effective stress ($p'$) and $p_a$ is the
reference pressure taken as the atmospheric pressure ($p_a = 101$ kPa). Young’s modulus increases
with increasing stress level and decreasing void ratio (i.e., denser soil). For the plastic aspect, there
are two expressions to determine the mobilized dilation angle ($\psi$) and friction angle ($\phi$) by the soil
state parameter, given in Equation 5-3 and Equation 5-4, respectively.

$$\tan \psi = A \left(1 - e^{\text{sign}(\Psi) \times |\Psi|} \right)$$  \hspace{1cm} \text{Equation 5-3}$$

$$\tan \phi = \tan \phi_c + \tan \psi$$  \hspace{1cm} \text{Equation 5-4}$$
where $A$ is a scaling factor, $n$ is a factor controlling the curve shape, $m$ is a factor mainly influencing the curve shape in the positive state parameter and $\phi_c$ is the critical state friction angle.

In contrast to the advanced model used for sand, a simple Tresca model is used for clay. There is only one plastic parameter, undrained shear strength ($s_u$), in the model to determine the yielding surface. The elastic parameter, Young’s modulus, is assumed to be proportional to the shear strength ($E = 500 \times s_u$). The stiffness ratio ($E/s_u$) is within the commonly adopted range (Hossain et al., 2005). To keep the fully undrained assumption for clay, Poisson’s ratio is chosen at 0.49 to prevent volumetric change while the numerical analysis is kept stable.

The CSMC model along with the LDFE/RITSS method for CPTs in layered soil, including a sand layer, has been verified with centrifuge test data (Xie et al., 2020a and 2020b). The good agreement shown in the verification can provide confidence in the soil model and the numerical results presented in this study. The soil parameters are calibrated for Toyoura sand and Kaolin clay and provided in Table 5-1.

5.3 NUMERICAL MODEL AND PARAMETRIC STUDY

Figure 5-1 graphically defines the problem and includes all important variables. The cone penetrates the layered soil from the soil surface to depth $d$, measured from the cone shoulder to the soil surface. The reaction force acting on the tip will be calculated and recorded through LDFE analysis. The controlling variables on the soil strata dimensions are layer thicknesses for the top clay layer ($H_t$) and the interbedded sand layer ($H_s$). The controlling variables on the soil strength/stiffness are the undrained shear strength ($s_{ut}$ and $s_{ub}$ for the top and bottom clay layers, respectively) for the clay and the relative density ($I_D$) for the sand.

Table 5-2 shows the study plan designed by varying the four controlling variables ($H_s/D$, $s_{ut}$, $s_{ub}$ and $I_D$). The top clay layer thickness is set as a constant at 20D to achieve the fully mobilized resistance in that layer (Ma et al., 2016). The values of the undrained shear strength for the clays and the relative density for the sand are chosen to cover the commonly encountered range. This
paper focuses on the scenario where the top clay layer and bottom clay layer have different undrained shear strengths, following the study by the same authors for a sand layer embedded in a uniform clay (Xie et al., 2020a)

The mesh setup is shown in Figure 5-2. To save numerical cost, the cone penetration is simplified as an axisymmetric problem. The bottom clay layer is 25D thick to avoid boundary effects, and the radius of the domain is the same as the total height (sum of the thicknesses of the three layers). The lateral sides are set as roller conditions to eliminate horizontal movements with free vertical movements. The bottom boundary is set as the hinge condition to eliminate both vertical and horizontal movement. A six-node triangular element with three Gauss points is deployed. To avoid numerical difficulty, the cone shoulder is buried in the soil at the start of the analysis. A mesh refined zone is added adjacent to the cone and moves with the cone tip to improve the accuracy in the localized high stress/strain area.

5.4 RESULTS AND DISCUSSION

5.4.1 Typical resistance profile

Figure 5-3 shows the resistance profiles of two scenarios: (a) Case A - the top clay is stiffer ($s_{ut} = 80$ kPa) than the bottom clay ($s_{ub} = 10$ kPa); (b) Case B - the top clay layer ($s_{ut} = 10$ kPa) is softer than the bottom clay ($s_{ub} = 80$ kPa). Depending on the noticeable changes in the profile gradient, both profiles are coloured in three zones when the cone penetrates in the sand layer: (I) blue zone - the resistance profile increases linearly and sharply when the cone penetrates shallowly into the sand layer, starting from the kink point registered in the top clay layer above the clay-sand interface; (II) red zone - there is another linear increase in the resistance profile while the increasing gradient is lower than that of Zone I, leading to the peak resistance at the end of this stage; (III) orange zone – the cone resistance decreases after reaching the peak and then converges to the ultimate resistance in the bottom clay.
To investigate the evolution of soil failure mechanisms during cone penetration in clay-sand-clay soils, there are 10 stages denoted for Case A and Case B in Figure 5-3. For Case A: Stage $A_1$ - the cone resistance is stabilized at its ultimate resistance in the top clay; Stage $A_2$ - the resistance decreases after the stabilized resistance is reached and before entering the sand layer; Stage $A_3$ - the kink point is formed right above the clay-sand interface; Stage $A_4$ - the cone shoulder goes across the original location of the clay-sand interface while the resistance increases linearly; Stage $A_5$ - the resistance reaches a turning point at which the resistance increases at a lower rate than that in Stage $A_4$; Stage $A_6$ - the resistance is increasing at the reduced rate; Stage $A_7$ - the peak resistance is reached in the sand layer; Stages $A_8$ to $A_9$ and $A_{10}$ - the resistance gradually decreases until reaching stable resistance in the bottom clay. There are some numerical oscillations at the end of the simulation near the sand-clay interface. For Case A, the cone in the top clay layer reaches its ultimate resistance at $12D$ (i.e., 920 kPa, resulting in a bearing factor of 11.5). Then, the cone resistance decreases with further penetration until the kink point at $19.3D$ above the clay-sand interface. After the kink point, the cone resistance starts to increase sharply due to the influence of the approaching sand layer.

Correspondingly, 10 stages for Case B are chosen in a similar manner to Case A. The differences observed in the cone resistance profiles between Case A and Case B are as follows: (a) there is no resistance reduction in the top clay layer after the ultimate resistance is reached at Stage $B_1$ ($d/D = 12$); (b) after the kink point in the top clay (i.e., Stage $B_3$), the sharp increase in resistance lasts shorter to Stage $B_5$; (c) the resistance increases linearly at a reduced rate that lasts longer between Stage $B_5$ and Stage $B_7$; (d) the rate of resistance increase between Stages $B_5$ and $B_7$ is higher than that in the corresponding period between Stages $A_5$ and $A_7$; (e) the peak resistance at Stage $B_7$ is higher than that at Stage $A_7$, which shows that the bottom clay has more influence on the cone peak resistance in sand; (f) after the peak resistance, the cone resistance decreases more sharply from Stage $B_7$ to Stage $B_{10}$ than that from Stages $A_7$ to $A_{10}$.
The corresponding penetration depths of d/D at the different stages are listed in Table 5-3

5.4.2 Flow mechanism

5.4.2.1 Case A: the sand layer interbedded in stiff top and soft bottom clays

Figure 5-4 shows the flow mechanism from stage A₁ to A₁₀ for Central Case A in Figure 5-3. In the top clay, a cavity expansion failure is observed at Stages A₁ and A₂ until the cone tip touches the clay-sand interface at Stage A₃. The localized failure mode results in the constant ultimate cone resistance being reached, and the resistance only depends on the local shear strength in the clay. As the clays are assumed to be uniform in the study, the cone resistance should be constant from Stage A₁ as long as the cone does not sense the new layer. At Stage A₂, the reduction from the ultimate constant resistance suggests that the cone has sensed the new layer, but the change is not noticeably reflected in the flow mechanism. From Stages A₃ to A₅, the cone passes through the clay-sand interface and corresponds to Zone I in Figure 5-3 (a). As the magnitude of the stress being mobilized in the sand layer is much higher than that in the clay, the penetration in the sand increases the resistance at a high gradient, resulting in a sharp increase over the three stages from A₃ to A₅, and the kink is registered at Stage A₃. Soil movement is disconnected by the layer interface. The clay is squeezed sideways between the cone face and the interface, while the sand near the cone face is pushed downwards. The clay-sand interface sags from its original location by 0.18D at Stage A₃, 0.25D at Stage A₄ and 0.35D at Stage A₅. From Stages A₅ to A₇, the cone is fully embedded in the sand, and the cone resistance increases with the penetration depth at a lower rate (Zone II in Figure 5-3 (a)) than that observed from Stages A₃ to A₅ (Zone I in Figure 5-3 (a)). At Stage A₆, the typical cavity expansion is observed again, similar to that at Stage A₁. This implies that the cone merely senses influences from the surrounding clay other than the overburden pressure at the stage. However, at Stage A₇, the soil displacement exhibits more sideways movement around the cone shoulder, and the displacements bend downward below the cone face. This minor change in the flow mechanism implies the sensing of the bottom soft clay. After Stage
A7, the influence of the bottom soft clay becomes increasingly noticeable with resistance continuing to decrease and then converging to the ultimate resistance in the bottom clay (Zone III in Figure 5-3 (a)). Hence, the peak resistance is observed at Stage A7. From Stages A8 to A10, the soil displacements continue to bend downward as the cone approaches the bottom layer. The size of the mobilized sand continues to decrease. This reduction in the mobilized sand causes the decreasing cone resistance in Zone III in Figure 5-3 (a). The sand-clay interface sags extensively by 1D, compared to only 0.18D for the clay-sand interface, as the bottom clay is much softer than the top clay (i.e., su/sub = 8), which results in a kink formed in the profile far from the original location of the interface (e.g., dk/b > 1D).

5.4.2.2 Central Case B: the sand layer interbedded in soft top and stiff bottom clays

The soil flow mechanism over the 10 stages for Central Case B is shown in Figure 5-5. Many characteristics observed in Case A can also be noted for Case B, including (1) cavity expansion when the cone penetrates in the top clay before the cone tip touches the clay-sand interface (Stages B1 and B2); (2) cavity expansion when the cone becomes fully embedded in the sand without sensing the bottom clay (Stages B5 and B6); and (3) displacements bending downwards due to attraction by the soft bottom layer (Stages B7 to B9). The similarity of the flow mechanisms between Case A and Case B explains their similar scopes of the resistance profile (three-stage development as demonstrated in Figure 5-3 (a) and (b)).

However, there are noticeable differences between the cone resistance profiles of Case A and Case B, as mentioned above, which can be explained in more detail using the mechanisms displayed in Figures 5-4 and 5-5. By comparing the flow mechanisms of Stages B3 to B5 and those of Stages A3 to A5, the squeezing effect is much stronger for Case B than for Case A, as the top clay in Case B is softer. The sagging of the clay-sand interface is smaller (or nearly no sagging) for Case B, as the sand layer is much stiffer than the top clay layer. The minimal sagging of the layer interface for Case B makes the cone tip touches the sand layer early, resulting a larger dk/b/D for Case B than
Case A. When the cone passes the sand-clay interface after the peak resistance in the sand layer (i.e., Zone III in Figure 5-3), the sagging of the sand-clay interface is smaller for Case B (i.e., Stages B_7 to B_{10} in Figure 5-5) than that for Case A (i.e., Stages A_7 to A_{10} in Figure 5-4). This makes the kink registered in the bottom clay closer to the initial location of the sand-clay interface, resulting in a smaller d_{kb} in Case B (d_{kb}/D < 1). At Stage B_{10}, the cone becomes fully embedded into the bottom clay layer, and soil flow only occurs in the bottom clay layer; hence, the cone reaches ultimate resistance in the bottom clay.

5.4.3 Further discussion on the profile characteristics

The above discussion revealed the soil flow mechanisms around the cone for both Case A and Case B and their linkage to the features of the cone resistance profiles. In this section, further discussion on two particular questions is provided.

(i) Why does the reduction from the ultimate resistance in top clay only appear in Case A? The phenomenon can be explained by the relative stiffness between each layer in the clay-sand-clay soils. Figure 5-6 shows the mean stress for Cases A and B. The stress bubble decreases from Stages A_1 to A_2 but increases slightly from Stages B_1 to B_2. For both cases, the cone sensing zones (indicated by the outlier of the stress bubble) are fully contained in the top clay at d/D = 12, while they expand into the sand layer at d/D = 16.5. When the top clay layer is stiff (s_{ut} = 80 kPa for Case A), the sand in the sensing zone responds with high stress in the 50 kPa-100 kPa band, while the sand in the sensing zone at Stage B_2 responds with low mean effective stress in the 10 kPa-20 kPa band when the top layer is soft (s_{ut} = 10 kPa for Case B). As suggested by Equation 5-2, the sand stiffness (E_s) is a function of the mean effective stress and void ratio. In Figure 5-6, a point is selected to represent the sand stiffness right below the cone, measured one diameter away from the central line and one diameter below the initial clay-sand interface. Young’s modulus of the point is calculated by Equation 5-2: for Case A, E_s = 10.26 MPa at Stage A_1 and 20.79 MPa at Stage A_2; for Case B, E_s = 4.68 MPa at Stage B_1 and 10.53 MPa at Stage B_2. The stiffness of the top clay (E_c)
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is calculated by its constant ratio to the undrained shear strength \((E_c/s_u = 500)\): for Case A, \(E_c = 80\) kPa \(\times\) 500 = 40 MPa; for Case B, \(E_c = 10\) kPa \(\times\) 500 = 5 MPa. The mobilized Young’s modulus of the sand (10.53 MPa) increases to larger than that of the top clay (5 MPa) at Stage B2, resulting in the cone resistance in the clay continuing to increase. However, the sand stiffness \((E_s = 10.26\) MPa at Stage A1 and 20.79 MPa at Stage A2) is below the top clay stiffness \((E_c = 40\) MPa). The sand layer behaves as a soft layer below the cone, resulting in reduction in the cone resistance. This phenomenon remains until the cone penetrates into the sand with high stress being mobilized.

(ii) What are the factors influencing the kink distances \((d_{kt} and d_{kb})\)? The kink distances are directly related to the deformation of the layer interfaces. As seen in Figures 5-3 to 5-5, the kink points in the top clay layer occur when the cone tip touches the deformed clay-sand interface. The kink points in the bottom clay layer occur when the cone shoulder detaches from the deformed sand-clay interface and enters the bottom clay layer. The interface deformation is influenced by the stiffness ratio of the adjacent layers.

For the kink point in the top clay, where \(E_{ct}/E_s > 1\) (\(E_{ct}\): stiffness of the top clay) for Case A, the clay-sand interface tends to sag downwards to the softer layer underneath (Figure 5-4), resulting in the kink appearing later. In Case B, where \(E_{ct}/E_s < 1\), there is minimal interface sagging but apparent squeezing of the clay (Figure 5-5), resulting in the kink appearing early. Thus, a lower \(d_{kt}\) is observed for Case A than that for Case B. For the kink point in the bottom clay, a similar phenomenon is observed. Where \(E_s/E_{cb} > 1\) (\(E_{cb}\): stiffness of the bottom clay) for Case A, the sand-clay interface sags more into the bottom clay as the underneath clay layer is softer, resulting in a higher kink distance \(d_{kb}\). Where \(E_s/E_{cb} < 1\) for Case B, less interface sagging results, hence less distance \(d_{kb}\). However, since the cone diameter is only 0.0357 m, the variation in the kink distance from the influential factors can be fairly small (for example, the difference in \(d_{kt}\) between Case A and Case B is 0.36 cm, i.e., \(~0.1D\)). Therefore, the study suggests constant values for \(d_{kt}/D\) and
$d_{k}/D$ of 0.75 and 1, respectively. They are the averages values of Case A and Case B. It can be confirmed by profiles in the later section that the constant values are reasonably accurate.

(iii) What are the differences in the flow mechanism of the cone in sand layer sandwiched by same clays and by different clays? When a sand layer is sandwiched by two clay layers with different strengths, the three layers work systematically to affect the flow mechanisms in the sand layer. As discussed early, when the top clay is stiffer than the bottom clay layer, both top stiff clay and sand layers act on one soft bottom clay layer. Thus, once the cone enters the sand layer, the stresses in sand are mobilized to a high level near the top layer interface (C-S) due to the top stiff clay, which results a stiffer sand layer being mobilized. Due to the soft bottom clay layer, the soil movements are attracted to the soft soil very early, hence the cone peak resistance is registered near the top C-S interface. When the top clay is softer than the bottom clay layer, the stiff bottom clay acts as a strong foundation for the top clay and the sand layer. During the cone penetration in the sand layer, the stresses in sand gradually increases until near the bottom layer interface, where the cone senses the bottom stiff clay and a peak resistance is registered. However, when the cone penetrates in a sand layer sandwiched between two clay layers with the same strength, the flow mechanism in the sand layer is influenced fairly equally by the two clay layers. Thus the cone resistance profile in the sand layer is more smooth and rounded (i.e. without sharp peak) and the peak resistance is more or less appearing at the mid-point of the sand layer.

5.4.4 Effect of the undrained shear strength of clay, $s_{ut}$ and $s_{ub}$

Figure 5-7 shows the effects of the clay undrained shear strength on the cone resistance profile. In Figure 5-7 (a), where the top clay layer is stiffer than the bottom clay ($s_{ut}/s_{ub} > 1$), the shear strength of the top clay is kept constant at 80 kPa while varying the shear strength of the bottom clay from 10 kPa ($s_{ut}/s_{ub} = 8$) to 60 kPa ($s_{ut}/s_{ub} = 1.33$). The cone resistance reduction in the top clay after its ultimate value ($q_{clayult}$ at $d/D = 12$) occurs for all cases in Figure 5-7 (a). The reduction decreases with the decreasing ratio of $s_{ut}/s_{ub}$. This means that the bottom soft clay can impact the cone
resistance in the top clay layer across the 5D-thick medium-dense sand layer ($I_D = 60\%$). The influence reaches up to 6D above the sand layer. Similarly, in the sand layer, the bottom soft clay influences the cone resistance, as the increasing measured peak resistance with the bottom clay stiffness implies. Therefore, the three-layer soils act as a system. The bottom soft layer can affect the cone resistance profile in both the top clay and middle sand layers. The clay strength ratio of $s_{ul}/s_{ub}$ also affects the shape of the cone resistance profiles in the sand layer. After the peak resistance, the reduction in resistance is sharp for $s_{ul}/s_{ub} \geq 4$, while the reduction is more progressive for $s_{ul}/s_{ub} \leq 2$. The location of the peak resistance moves closer to the midpoint of the sand layer when $s_{ul}/s_{ub}$ varies from 8.0 to 1.0. The difference is caused by the bottom clay stiffness. As shown by the flow mechanism (Figure 5-4), the peak resistance is reached when the cone senses the bottom clay. The softer the bottom clay is (i.e., higher $s_{ul}/s_{ub}$), the earlier it is sensed, hence the earlier appearance of the peak resistance and lower peak resistance. In Figure 5-7 (b), the effect of clay strength is shown for $s_{ul}/s_{ub} < 1.0$, where the top clay layer is softer than the bottom clay layer, and the undrained shear strength of the top clay layer is kept constant at $s_{ul} = 10$ kPa while varying that of the bottom clay layer from 20 kPa ($s_{ul}/s_{ub} = 0.5$) to 80 kPa ($s_{ul}/s_{ub} = 0.125$). Due to the stronger bottom clay layer, no resistance reduction is observed in the top clay layer. Combined with the observations from Figure 5-7 (a), it can be seen that when $s_{ul}/s_{ub} \leq 1.0$, all resistance profiles in the top clay merge together (see the inserted graph in Figure 5-7 (b)). This means that the soil failure mechanisms are contained in the top clay when the bottom clay is strong enough (i.e., $s_{ul}/s_{ub} \leq 1.0$) to provide support for the embedded thin sand layer. The peak resistance in the sand layer increases with the decreasing strength ratio of $s_{ul}/s_{ub}$, and the location of the peak resistance deviates from the midpoint of the sand layer towards the bottom clay when $s_{ul}/s_{ub}$ decreases from 0.5 to 0.125.

5.4.5 Effect of sand layer thickness, $H_s$

Figure 5-8 shows the effects of normalized sand layer thickness on the resistance profiles for both Case A ($s_{ul}/s_{ub} > 1.0$, Figure 5-8 (a)) and Case B ($s_{ul}/s_{ub} < 1.0$, Figure 5-8 (b)). It is apparent that,
in both cases, the peak resistance in sand increases with increasing sand layer thickness. The resistance reduction in the top clay from its ultimate resistance is observed for Case A in Figure 5-8 (a) but not for Case B in Figure 5-8 (b) for the same reason stated above. The magnitude of resistance reduction decreases with increasing sand layer thickness, with the highest reduction for \( H_s/D = 3 \) at 13.3\%. This means that the sand layer and the bottom clay layer act together to influence the resistance in the top clay layer. For Case A, when the cone penetrates into the thick sand layer \( (H_s/D = 10) \), the soil failure mechanism is fully retained in the sand layer for 2.4D (from Stage A5 at 20.3D to Stage A6 at 22.7D, Zone II in Figure 5-3 (a)). However, when the sand layer is as thin as \( H_s/D = 3 \) and 5, Zone II disappears where Zone I (i.e., resistance increasing sharply) is followed by Zone III (i.e., resistance decreasing sharply) directly. Clearly, when the sand layer is too thin (i.e., \( H_s/D < 7 \)), there is not enough volume to fully develop the flow mechanism in the sand layer without sensing the bottom clay.

However, for Case B of \( s_u/s_u^* < 1.0 \) in Figure 5-8 (b), the three zones defined in Figure 5-3 (b) can always be observed in all cases of \( H_s/D = 3, 5 \) and 10. This is because the stronger bottom clay provides enough support to the top clay and sand layers; hence, the cone resistances in the top clay and Zone I are not affected by the sand layer thickness. The rate of cone resistance increase in Zone II decreases with sand layer thickness. This is because the bottom stronger clay has less influence on the cone resistance in the zone when the sand layer is thicker. However, the rate of decrease in resistance in Zone III is similar for all cases, which means that the bottom clay has the dominant effect at the stage. This effect can be confirmed by Figure 5-7 (b), where the rate of decrease in resistance in Zone III increases with increasing strength of the bottom clay.

### 5.4.6 Effect of sand relative density, \( I_D \)

Figure 5-9 shows the impact of the relative density of the thin sand layer on the resistance profiles. The cone penetration curves show characteristics similar to those discussed above regarding Figures 5-3, 7 and 8, such as: (i) in the top clay layer, the reduction from the ultimate resistance in
the top stiff clay for Case A; (ii) in the sand layer, the sharp peak resistance for Case A ($s_{ut}/s_{ub} = 8.0$) but progressive peak resistance for Case B ($s_{ut}/s_{ub} = 0.125$); (iii) in the sand layer, the resistance profile having two development zones for Case A and three development zones for Case B. As in Case U (a sand layer in uniform clay, $s_{ut} = s_{ub}$, as in Figure 5-9 (c)), the cone peak resistance in sand increases with its relative density from loose (I$_D = 30\%$) to medium dense sand (I$_D = 60\%$) for both Cases A and B. However, the peak resistance in the very dense sand (I$_D = 90\%$) shows a different trend: for both Cases A and B, the resistance profiles show the three development zones (Zone I: sharp increase in resistance; Zone II: gradual increase in resistance; Zone III: decrease in resistance), and the peak resistances are 10.3% and 10.7% lower than that in the medium dense sand (I$_D = 60\%$) for Cases A and B, respectively. After the peak, the cone resistance decreases to the ultimate resistance in the bottom clay once the cone enters the bottom clay layer, which is similar to those observed for I$_D = 30\%$ and 60\%.

Although the peak resistance in the very dense sand of I$_D = 90\%$ is expected to be the highest based on the observations for Case U, the thin sand layer behaves differently for Cases A and B when $s_{ut} \neq s_{ub}$. Figure 5-10 shows the stiffness development for Case A with I$_D = 60\%$ and 90\%. The sand stiffness is a function of the sand relative density and the mean effective stress, as shown in Equation 5-2. By comparing the sand stiffness of I$_D = 60\%$ and 90\%, due to the high density of I$_D = 90\%$, the stiffness develops more quickly towards the bottom soft clay layer. The stiffness ratio of $E_s/E_{cb}$ is higher for I$_D = 90\%$ than that for I$_D = 60\%$. This higher stiffness ratio can encourage punching-through failure immediately after the cone enters the sand layer and make the thin stiff sand behave like a brittle beam.

The effect of punching-through failure can be seen more clearly in the normalized vertical displacement graphs for both I$_D = 60\%$ and 90\% in Figure 5-11. In Figure 5-11, substantial downward movement is observed from $d/D = 20$. Over the four depths, there are more downward movements in I$_D = 90\%$ than in I$_D = 60\%$. The movement of the whole punched block substantially
releases the stress and transfers the loading to the bottom clay. The earlier reliance on the bottom soft clay results in no development of cavity expansion in the sand layer, hence the lower cone resistance in the development Zone II and the lower peak resistance of $I_D = 90\%$ than that of $I_D = 60\%$. Thus, the lower cone peak resistance in the very dense sand is due to the ‘brittleness’ of the thin sand layer. The difference in the strengths of the top and bottom clay layers encourages the brittle behaviour of the sand layer. In Case U, as shown by Figure 5-9 (c), when $I_D = 75\%$ and $90\%$, the rate of resistance increase before the peak is lower than that of $I_D = 60\%$, although the peak reduction is not obvious. This could be a sign of punching through failure. For Case U, the peak resistance will be nearly constant when $I_D > 60\%$; for Case A and Case B, the peak resistance will decrease when $I_D > 60\%$.

5.5 SUGGESTIONS FOR INTERPRETING CPT DATA

5.5.1 Interpretation of the clay layers

In this study, the resistance in the top clay layer reaches the ultimate resistance at 12D. The ultimate resistance remains until the cone senses the impact of the bottom layers at 6D above the sand layer for Case A, resulting in a reduction in the resistance. For Case B, the resistance in the top clay starts to change due to the bottom layers until the top kink. When the cone penetrates into the bottom clay, the ultimate resistance is quickly reached in the bottom clay. Therefore, the ultimate cone resistance can be directly utilized to interpret the clay layers by the existing method derived for single-layer clay.

5.5.2 Relative density of the sand layer

As discussed above, the cone resistance in the sand layer is influenced by both the top and bottom clays. Based on all the results from LDFE/RITSS analysis, Figure 5-12 shows a regression term between the peak resistances and the factored undrained shear strengths of both clay layers. The regression term is designed to include the impact from the surrounding clay ($s_{ut}$ and $s_{ub}$), the relative density of the sand ($I_D$) and the stress level of the sand layer ($p'_0$). The dataset where the sand layer
is embedded in uniform clay (Case U) is also included in Figure 5-12. As the top clay layer was varied in the study of Case U, the regression term in Figure 5-12 also includes the normalized top layer thickness ($H_t/20D$).

With the two soil flow modes for Case A and Case B as discussed above, two linear functions can be fitted for $s_{ut} > s_{ub}$ and $s_{ut} < s_{ub}$, respectively, with high $R^2$. The two lines intersect at $\alpha s_{ut} + (1-\alpha)s_{ub} = 31.6$ kPa. It can be noted that the relationship for the sand in uniform clay follows the $s_{ut} < s_{ub}$ line when the factored clay shear strength is less than 31.6 kPa but follows the $s_{ut} > s_{ub}$ line when the factored clay shear strength is more than 31.6 kPa.

Formulas can be proposed to interpret the sand relative density by rearranging the regression linear functions as follows:

$$I_0(\%) = 100 - \frac{q_p^2 s_{ut} s_{ub}^2 (H_t / 20D)^5 (\gamma_s D)^2}{C_1(\alpha s_{ut} + (1-\alpha)s_{ub}) - C_2} p_0^6$$

Equation 5-6

where $\alpha = 0.3$, $p_0'$ is the initial mean effective stress at the middle point of the sand layer and $\gamma_s$ is the sand effective unit weight in KN/m$^3$. For Case A ($s_{ut}/s_{ub} > 1.0$), the factors are $C_1 = 7269.7$ and $C_2 = 215331$, and for Case B ($s_{ut}/s_{ub} < 1.0$), $C_1 = 518.89$ and $C_2 = 1640$.

**5.5.3 Identification of the very dense thin sand layer**

The inconsistency in increasing peak resistance with relative density introduces complexity to the data interpretation. The different profile mode (progressive peak) for the very dense sand could be utilized as evidence to note the very high density in Case A (see Figure 5-9 (a)): when the top clay is much stiffer than the bottom clay layer (i.e., $s_{ut}/s_{ub} \geq 4$ in Figure 5-7(a)), a sharp peak resistance is exhibited in loose to medium dense sand layers, while a progressive peak resistance is observed for very dense sand. This indicates that the sand relative density cannot be directly estimated by the peak resistance, even just qualitatively. Therefore, careful sampling for laboratory tests could be considered as an option when very dense sand is noted.
5.6 CONCLUSION

The paper reports LDFE results for the CPT in a thin sand layer sandwiched between clay layers with different undrained shear strengths. Two modes of the resistance profiles are defined for a stiff top clay with soft bottom clay and a soft top clay with stiff bottom clay. These two profiles have been explained by their flow mechanism and the mean stress mobilization. The common characteristics in the two profiles are the three-stage resistance development: (1) the cone resistance sharply increases when the cone penetrates through the clay-sand interface; (2) the cone resistance continuously increases at a reduced rate until reaching the peak resistance, with the cone merely sensing the bottom clay; and (3) the resistance eventually decreases to converge with the ultimate resistance in the bottom clay. However, due to the various clay strengths around the sand layer, with different “support” being provided by the bottom clay, the resistance profile registers different peak resistances at different locations. A stronger bottom clay contains the flow mechanism to the top layers, resulting in the peak resistance being registered closer to the sand-clay interface. The change of upper clay strength alters the stress development in the shallow penetration in the sand layer, which results changes in mobilized stiffness of the sand and hence influence the flow mechanisms of the sand layer. Additionally, when the bottom clay is soft relative to the top clay, the soft bottom provides less support to the sand layer, and the impact of the bottom layer can be transferred through a 5D-thick medium sand layer. Reduction from the ultimate resistance of the top clay is observed at less than 6D above the sand layer when the cone approaches the embedded sand layer.

Using the collected numerical data, a relationship of the peak resistance in the sand with the clay undrained shear strengths, sand density and initial stress level are established. A linear function and two sets of coefficients in the function are provided to predict the relative density for the sand layer embedded in stiff-over-soft clay and soft-over-stiff clay. As the formulas were derived based on the numerical results in this study, they need to be taken with caution when extrapolating any data outside of the study range. The results show that when the sand layer is of very high density
(i.e., 90%), the peak resistance can be even smaller than that of less dense sand. This result adds complexity to interpretation of the CPT data. Sampling for laboratory tests is suggested for these conditions.

The study highlights the necessity of adopting an advanced sand model in numerical analysis to capture the complex behaviour of granular materials. More physical test data should be welcomed to further verify the results and build comprehensive guidelines for interpretation purposes.

5.7 REFERENCE


XIE, Q., HU, Y. & CASSIDY, M. J. (2020b). Interpretation of cone penetration test data in sand over clay in centrifuge test. To be submitted


Figure 5-1. Schematic graph of the CPT in a clay-sand-clay soil

Figure 5-2. Mesh setup for the CPT in clay-sand-clay
Figure 5-3. CPT resistance profile in clay-sand-clay soils ($H_i/D = 20$, $H_s/D = 10$ and $I_D = 60\%$ in Table 5-2): (a) Case A: $s_{ut}/s_{sub} = 8$ and $s_{ut} = 80$ kPa; and (b) Case B: $s_{ut}/s_{sub} = 0.125$ and $s_{ut} = 10$ kPa

(C-S refers to the clay-sand interface, while S-C refers to the sand-clay interface)
Figure 5-4. Soil flow mechanism from Stages \( A_1 \) to \( A_{10} \) for Central Case A \( (H_t/D = 20, H_s/D = 10, s_u/s_{ub} = 8, s_u = 80 \text{ kPa and } I_D = 60\%, \text{ Table 5-2}) \)
Figure 5-5. Soil flow mechanism from Stages B_1 to B_{10} for Central Case B (H_t/D = 20, H_s/D = 10, \(s_{ut}/s_{ub} = 0.125\) and \(I_D = 60\%\) in Table 5-2)
Figure 5-6. Mean stress contours at (a) Stage A₁, (b) Stage A₂, (c) Stage B₁, and (d) Stage B₂ (MS refers to the mean stress)
Figure 5-7. Effect of undrained shear strength $s_{ut}$ and $s_{ub}$ on the cone resistance profile ($H_b/D = 5$, $I_D = 60\%$, Group I and Group IV in Table 5-2), (a) Case A: $s_{ut} = 80$ kPa and $s_{ub} = 10, 20, 40$ and $60$ kPa and (b) Case B: $s_{ut} = 10$ kPa and $s_{ub} = 20, 40, 60$ and $80$ kPa
Figure 5-8. Effect of sand layer thickness $H_s/D$ on the cone resistance profile ($I_D = 60\%$, Group I and Group IV in Table 5-2), (a) Case A: $s_{ut} = 80$ kPa and $s_{ub} = 10$ kPa and (b) Case B: $s_{ut} = 10$ kPa and $s_{ub} = 80$ kPa
Figure 5-9. Effect of sand relative density $I_D$ on the cone resistance profile ($H_c/D = 5$, Group I and Group IV in Table 5-2; data from another study (Xie et al., 2020a)), (a) Case A: $s_{ut} = 80$ kPa and $s_{ub} = 10$ kPa (b) Case B: $s_{ut} = 10$ kPa and $s_{ub} = 80$ kPa and (c) Case U: Case B: $s_{ut} = 20$ kPa and $s_{ub} = 20$ kPa
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Figure 5-10. Stiffness development in the sand layer for Case A ($s_{nt} = 80$ kPa and $s_{ub} = 10$ kPa) with $I_D = 60\%$ and $90\%$
Figure 5-11. Vertical normalized displacement comparison between $I_D = 90\%$ and $60\%$ for $s_{ut} =$ 80 kPa and $s_{ub} =$ 10 kPa
Figure 5-12. Relationship between the peak resistance in sand and the factored undrained strength of the surrounding clay ($\alpha = 0.3$)

$$y = 7269.7x - 215331$$

$$R^2 = 0.962$$

$$y = 518.89x - 1640$$

$$R^2 = 0.941$$
Table 5-1. Basic engineering properties of Toyoura sand and its parameters in the CSMC model

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<th>$e_{\text{min}}$</th>
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<th>m</th>
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Table 5-2. Summary of the parametric study plan

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<th>Category</th>
<th>Group</th>
<th>Top clay undrained shear strength, $s_u$ (kPa)</th>
<th>Normalized sand layer thickness ($H_s/D$)</th>
<th>Sand relative density, $I_D$ (%)</th>
<th>Bottom clay undrained shear strength, $s_u$ (kPa)</th>
<th>Notes</th>
</tr>
</thead>
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<td>Stiff-over-soft clay (Case A)</td>
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<td>10</td>
<td>Effect of the sand layer thickness</td>
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<td></td>
<td>II</td>
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<td>5</td>
<td>30, 60, 90</td>
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<td>Effect of the sand relative density</td>
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<td>60</td>
<td>10, 20, 40, 60</td>
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<tr>
<td>Soft-over-stiff clay (Case B)</td>
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<td>3, 5, 10</td>
<td>60</td>
<td>80</td>
<td>Effect of the sand layer thickness</td>
</tr>
<tr>
<td></td>
<td>V</td>
<td>10</td>
<td>5</td>
<td>30, 60, 90</td>
<td>80</td>
<td>Effect of the sand relative density</td>
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<tr>
<td></td>
<td>VI</td>
<td>10</td>
<td>5</td>
<td>60</td>
<td>20, 40, 60, 80</td>
<td>Effect of the clay undrained shear strength</td>
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<tr>
<td>Central Case A</td>
<td></td>
<td>80</td>
<td>10</td>
<td>60</td>
<td>10</td>
<td>Results on the flow mechanism</td>
</tr>
<tr>
<td>Central Case B</td>
<td></td>
<td>10</td>
<td>10</td>
<td>60</td>
<td>80</td>
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Table 5-3. Penetration depths for the penetration stages

<table>
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<tr>
<th>Stage number</th>
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<tr>
<td></td>
<td>Case A</td>
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<tr>
<td>1</td>
<td>12</td>
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<td>9</td>
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</tr>
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<td>10</td>
<td>30.2</td>
</tr>
</tbody>
</table>
6 EFFECT OF LARGE DEFORMATION ANALYSIS FOR SITE INVESTIGATION TOOL - CPT IN LAYERED SOILS

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ABSTRACT

Cone penetration test (CPT) is regularly used during offshore site investigations to interpret soil stratification and soil characteristics due to its continuous penetration resistance profile. However, its use could be improved if better numerical methods to simulate its penetration could be developed. Finite element (FE) analysis, for instance, has the potential to provide insightful information on soil response and soil flow mechanisms. However, it is challenging to simulate CPT in layered soils, as the soil experiences extremely large strains around the cone and the simulation costs are high. In this study, the efficiency of using a partial large deformation FE (LDFE) approach was explored to examine the pre-embedment depth allowed for saving LDFE analysis cost. The LDFE analysis was conducted using the remeshing and interpolation technical with small strain (RITSS) method to model the large strain problem. Both soft-stiff-soft clays and clay-sand-clay soil were considered to study the thin stiff layer effect when it was sandwiched in soft clay. The LDFE/RITSS analysis compared a CPT penetrating from the soil surface with penetrations from a pre-embedded depth above the stiff layer. Pre-embedded small strain analysis was also conducted for comparison.

The results show that the small strain analysis underestimated the resistance in both clay and sand. For the partial LDFE analysis with pre-embedment in the top clay layer, the CPT response in the middle stiff clay layer could be well captured regardless of the initial pre-embedment depth. However, for the middle medium dense sand layer ($I_D = 60\%$), the pre-embedment depth needs to
have sufficient distance above it (10D, D is cone diameter) to capture the soil response in the sand layer correctly.

**KEYWORDS**

Cone penetration test; layered soil; large deformation finite element analysis; critical state sand model
6.1 INTRODUCTION

CPT is a commonly deployed test method for site investigations in both offshore and onshore fields. The test facility comprises a steel probe with 60° apex cone tip connected at the end of the shaft, rolling-down system and data collection program. The standard cone is 35.7 mm in diameter resulting in a 1000 mm\(^2\) basement area. It measures the reaction force acting on the cone tip, friction force along the shaft and the water pore pressure around the cone (i.e. piezocone). Among the three measurements, the cone tip resistance is of prime importance to interpret the engineering properties of soils by established correlations. Many researchers have worked on establishing the correlations, but a rigorous solution is extremely difficult due to the large strain and material nonlinearity associated with the CPT.

There are many studies on CPT in single layer soils using different approaches, including bearing capacity theory, cavity expansion theory, steady state deformation solution, finite element analysis and calibration chamber testing, as reviewed in (Yu and Mitchell, 1998). There are fewer studies on CPT in layered soil compared with those in single layer soils. Approaches being utilized for layered soils include cavity expansion theory (Mo et al., 2016), physical testing (Mo et al., 2015) and large deformation finite element (LDFE) analysis (Ma et al., 2017). It has been suggested that the LDFE numerical analysis methodology has many advantages over the other methods as it potentially provides additional insights into the relationships between soil characteristics and cone responses (Mo et al., 2015). However, there are difficulties in modelling the cone penetration numerically due to the large strain induced in the localized area around the probe and the non-linear nature of soil behaviours.

To overcome the problem, it is necessary to deploy advanced numerical techniques to track the development of the stress/strain around the probe. There are three common approaches capable of accounting for large strain deformation in geotechnical engineering: the implicit remeshing and interpolation technique by small strain (RITSS), the efficient Arbitrary Lagrangian–Eulerian (ALE) implicit method and the Coupled Eulerian–Lagrangian (CEL) method. All the approaches
have been applied to the cone penetration problem successfully and detailed comparison can be found in the other study (Wang et al., 2015). Thought the consistency in the three methods was shown, the RITSS method stood out by its remarkably smooth resistance profiles obtained. However, the analysis costs heavily in computational burden and time for cases when the cone penetrates deeply. For an example case in this study, it takes 82 hours to simulate a cone penetrating through a top clay layer of 20D and a middle sand layer of 5D, to achieve stable reading in the bottom clay on a PC with i7-4790 CPU at 3.60 GHz. 2,116 six-node triangular elements with three Gauss points are used for the analysis.

This paper is to study the CPT resistance profiles in layered soils, where a thin stiff layer is sandwiched in a soft clay. Both stiff clay and sand layers are considered as the thin stiff layer. The efficiency of LDFE analysis is explored for the cone penetration starting either from the soil surface or from different initial pre-embedment in the top soft clay layer. Furthermore, pre-embedded small strain analysis is also conducted as a comparison.

6.2 PROBLEM SETUP

Table 6-1 shows the set-up of the cone penetration in layered soft-stiff-soft soils. The stiff layer is either stiff clay with undrained shear strength $s_u = 60$ kPa or medium dense sand with relative density $I_D = 60\%$, while the soft layers are soft clay with $s_u = 20$ kPa. In this study, the top soft layer thickness, $H_t$, is 20D, which is thick enough to mobilize stable resistance within the layer. The middle thin stiff layer thickness, $H_s$, is 5D. For the full LDFE analysis, the cone shoulder is set at soil surface level (i.e. $d_{pre} = 0$) at the start of the analysis. For small strain and partial LDFE analysis, the cone is pre-embedded at the designed depth (i.e. $d_{pre} > 0$) to save LDFE computational cost. $d_s$ is the distance measured from the cone shoulder to the surface of the sand layer.

Table 6-1 summarizes the parametric study plan for a cone with different pre-embedment depths, $d_{pre}$ in small strain and LDFE analyses. For the convenience of discussion, the distance to the stiff layer, $d_s$, as shown in Figure 6-1, will be used in the following sections to represent the different
pre-embedment depths. For the small strain analysis, $d_{\text{pre}}$ ranges from 0 (i.e. at the soil surface) to 27D (i.e. 2D below the bottom of the stiff layer), to obtain the cone resistance at the various depths. There are more $d_{\text{pre}}$ designed in the stiff layer aiming to capture the peak cone resistance in the layer. In the study, the cone is assumed as fully smooth.

### 6.3 METHODOLOGY

The RITSS method (Hu and Randolph, 1998b) is chosen for the large deformation analysis. The method can be categorized as the ALE FE method. It starts with a series of small strain analysis with displacement control. Then the mesh is re-generated based on the deformed soil domain boundary from the small stain analysis results. The field variables (such as soil properties, strain and stress) are interpolated from the old deformed mesh onto the new mesh. Then another series of small strain analysis is performed on the new mesh. The process is repeated until the required cone penetration depth is achieved.

To model the stress-dependent behaviour of sand, the extended Critical State Mohr Coulomb (CSMC) model (Li et al., 2013) is deployed. The model modifies the Mohr Coulomb model by linking the state parameter, $\Psi$, under critical state concept to the variation of friction angle $\phi$ and dilation angle $\psi$. The state parameter is the difference between the in-situ void ratio and the critical state void ratio under the same stress level. In terms of the elastic parameters, the Young’s modulus is a function of the in-situ mean effective stress and void ratio to consider the stress-dependence. The model was specifically developed for the RITSS method, and it strikes a balance between accuracy and simplicity to keep the analysis numerically stable and efficient. For clay, a simple Tresca failure criterion is used. The clay Poisson ratio is chosen at 0.49 (i.e. close to 0.5 for saturated clay with no volume change and below 0.5 to keep numerical stability). Sand Poisson ratio is selected as 0.3.

The RITSS method and CSMC model are implemented in the AFENA package (Carter and Balaam, 1995), which is used in the study. H-adaptive mesh refinement is also implemented (Hu
and Randolph, 1998a) to optimize the mesh and minimize discretization errors. The method has been verified against to both physical data (Ma et al., 2014, Xie et al., 2019) and existing numerical solution (Ma et al., 2016) for cone penetration in layered clay and sand by full LDFE analysis, due to the page limit, it is not presented here.

6.4 RESULTS & DISCUSSION

6.4.1 Comparison between small strain and full LDFE Analysis

6.4.1.1 Soft-stiff-soft clays

Figure 6-2 shows the load-displacement results of small strain FE analysis for a pre-embedded cone at the soil surface (d_{pre}/D = 0), in the top clay layer (d_{pre}/D = 10 and 15), in the stiff clay layer (d_{pre}/D = 22) and in the bottom soft clay (d_{pre}/D = 26). In the Figure 6-2 (a), only the surface cone (d_{pre}/D = 0) reaches its ultimate capacity at d/D = 0.1, all other cones with d_{pre}/D > 0 have not reached their ultimate capacity even when d/D > 0.3. Such phenomenon was also observed by previous studies (De Borst and Vermeer, 1985). In Figure 6-2 (b), the cone resistance is normalized by the clay strength, q_c/s_u, which is known as bearing capacity factor of cone penetrometer. Same as Figure 6-2 (a), a constant bear capacity factor is only observed in the case where cone is pre-embedded at surface, the bearing capacity factor keeps increasing in the cases of deeply embedded cones. The calculated bearing capacity factors by the small strain analysis is well below the conventional average value, about 11.5 for cone at deep penetration depth. The different load-displacement responses are due to the different failure mechanisms between surface cone and the pre-embedded cone. The surface cone can generate a surface footing failure mechanism where stable shear failure lines reach the soil surface when the ultimate capacity is reached, while the pre-embedded cone follows the cavity expansion failure mode. By the cavity expansion mode, the displacement increment by the cone keeps pushing the soils laterally and vertically, hence the stress around the cone increases consistently. The bearing resistance of the cone at certain embedment depth can be identified when an ultimate value is reached within a small strain range, i.e. d/D ≤
0.3. Otherwise, the failure load is selected at the range of $d/D \leq 0.3$. In this study, the failure loads at both $d/D = 0.1$ and 0.3 are selected. It can be seen that, in Figure 6-2, the load-displacement curves at $d/D = 10$, 15 and 26 coincide with each other when the cone is pre-embedded in the top and bottom soft clay with the same undrained shear strength. Thus, the loads are a function of the local undrained shear strength, where the middle thin stiff layer has no influence on these three embedment cases. The different embedment does not change the resistance readings for $d_{pre}/D = 10$, 15 and 26. Small strain analysis cannot model the increasing resistance with depth (from $d_{pre}/D = 10$ to $d_{pre}/D = 15$).

The failure loads from small strain analysis at various pre-embedment depths from Figure 6-2 are compared with the full LDFE analysis result in Figure 6-3. The difference between the failure loads at $d/D = 0.1$ and 0.3 indicates the load is increasing with further displacement. The comparison between the loads from small strain and full LDFE analyses shows that, except for the surface cone, the small strain analysis always underestimates the resistance. In the soft clay, the small strain analysis underestimates the failure load by 44% when compared to the LDFE analysis. In the stiff middle clay layer, although the small strain analysis shows a peak load at the same location as the LDFE analysis, the peak load from small strain analysis is 31% lower than that of LDFE analysis. The underestimation emphasizes the large strain nature in CPT penetrations, since the small strain analysis could not mobilize continuous soil movement and soil layer deformations. Therefore, the small strain analysis cannot provide accurate prediction for the CPT load in the soft-stiff-soft layered clay except the cone is wished-in-placed at the soil surface.

#### 6.4.1.2 Clay-sand-clay

Figure 6-4 shows the small strain analysis results for clay-sand-clay soils at various pre-embedment depths. Similar to the results from the layered clay, there is no ultimate resistance observed except for the surface cone. The resistance in the sand layer ($d_{pre}/D = 22$) increases with displacement at a much higher rate than those in clays.
Figure 6-5 collates the CPT bearing resistance of small strain analyses at d/D = 0.1 and 0.3 in the clay-sand-clay soil. The full LDFE analysis result in the same layered soil is also displayed for comparison. A large difference in the results from small strain and large strain analyses is observed when the cone is embedded in the sand layer as the bearing resistance constantly increases with further displacement (see Figure 6-4). The small strain analysis shows the peak resistance at 23D, which is 1.5D below the peak resistance location from the LDFE analysis. Moreover, the peak resistance from the small strain analysis is 82% lower than that from the LDFE analysis. The deviation between the small strain and LDFE analysis becomes more significant in the middle sand layer when the stress-dependent model is deployed.

6.4.2 Comparison between partial and full LDFE analyses

6.4.2.1 Soft-stiff-soft clay

Figure 6-6 shows the resistance profiles of the partial LDFE analysis along with the full LDFE analysis. It is clear that, in partial LDFE analyses, as long as the cone is pre-embedded in the top soft clay (d_s/D ≥ 2), the cone resistance profiles in the stiff and the bottom soft layer are consistent with the full LDFE analysis results. When d_s/D = 2 (i.e. d_pre/D = 18), the partial LDFE result converges to the full LDFE results at d/D = 19.2 where the cone tip touches the stiff layer, since cone tip height is around 0.8D and the stiff layer is embedded at 20D. This implies that the deformation of the topside interface is minimal when the cone penetrates from soft to stiff clay.

The stress development is examined by comparing the case of partial LDFE analysis of d_pre/D = 15 with the full LDFE analysis of d_pre/D = 0, as shown in Figure 6-7. At d/D = 16, the cone in partial LDFE analysis only penetrates 1D, while it penetrates 16D in the full LDFE analysis. Thus the mean stress around the cone in the full LDFE analysis is fully developed and reaches its ultimate level, while that in the partial LDFE analysis of d_pre/D = 15 is still developing. After 3D of penetration in the partial LDFE analysis (i.e. d/D = 18), the stress contours in the partial LDFE analysis start to match those in the full LDFE analysis. Once the cone penetrates into the middle
stiff clay layer (d/D = 20 and 21.2 at its peak reading), the stress around the cone becomes identical from both partial and full LDFE analysis. Hence, the cone resistance profiles in the stiff layer and the bottom soft layer converge (Figure 6-6).

Regarding the computational cost, it takes around 23, 14, 12 and 10 hours to model the cone penetrating through the stiff clay layer from d_{pre}/D = 0, 10, 15 and 18 (corresponding to d_s/D = 20, 10, 5 and 2) respectively on the same machine as mentioned before. If the peak resistance in the thin stiff layer is the key concern, the pre-embedded cases can save substantially computational time.

6.4.2.2 Clay-sand-clay

Figure 6-8 includes the results of partial and full LDFE analysis for a cone clay-sand-clay soils. For all the partial LDFE analyses with the cone pre-embedded in the top clay layer (i.e. d_{pre}/D < 20), although the cone resistance profiles converge to that of the full LDFE analysis in the top soft clay layer after ~2D penetration, the cone resistance profiles in the middle sand layer are greatly affected by the initial pre-embedment, including both peak resistance value and its location. The only resistance profile in the sand layer coincides with the full LDFE results is when d_{pre}/D = 10 (i.e. d_s/D = 10).

Compared with the full LDFE analysis, the partial analysis underestimates the peak resistance in the sand layer by 6.2%, 12.5% and 19.9% for d_s/D = 5, 2 and 0 respectively. At the same time, the location of the peak resistance is moving downwards when the cone initial embedment is closer to the sand layer.

Figure 6-9 illustrates the mean stress development of the partial LDFE analysis with d_{pre}/D = 18 and full LDFE analysis at d/D = 19, 20 and their peak reading locations. It shows the mean effective stress is less mobilized through all the three stages by the partial LDFE analysis. At the stage of d/D = 19, the zone of developed stress by the full LDFE analysis is much larger than that by the partial LDFE analysis in both clay and sand. This means that the 1D penetration depth (from its
initial 18D to 19D now) could not fully develop the stresses in the soft clay layer. The non-fully mobilized stress in the clay layer impacts the stress development in the sand layer as the sand stiffness and strength are functions of stress level. Such impact can also be observed at the stage of d/D = 20 and at the peak resistance locations. At d/D = 20, the mobilized stress in the sand layer by the partial LDFE analysis is also lower than that by the full LDFE analysis. The lower stress could explain the difference in the peak resistance location. When the mobilized stress zone is smaller, the cone needs to be closer to the sand-clay interface to sense the softening effect from the soft clay layer to register the peak reading. The lower mobilized stress could also explain the decreasing peak resistance in sand layer when the cone is pre-embedded closer to the C-S interface in Figure 6-8. Sand is a stress-dependent material, different stresses being mobilized will result different initial state of the sand layer, hence varying the peak resistance in the layer. When the cone is embedded close to the C-S interface, the short distance could only mobilize low stresses, and less stiff sand, hence the peak resistance decreases.

From the above discussions, it is apparent that d_s/D = 10 can be considered as the minimum distance to the sand layer to obtain the fully mobilized stresses in the middle sand layer.

Regarding the computational cost, it takes 46 and 20 hours to model the cone penetrating through the sand layer from d_{pre}/D = 0 and 10 (correspondingly, d_s/D = 20 and 10) respectively. The computation cost is higher for the cone penetrating into the clay-sand-clay soil than that in the soft-stiff-soft clay. A long time is spent to solve the sand equations as the sand parameters are stress-dependent.

In summary, the stress developed in the top clay influences the mobilized stress in the middle layer. If the middle layer consists of stress-dependent material (i.e. sand), the strength condition of the layer is dependent on the different mobilized stresses, hence resulting different resistance profiles. When the middle layer material is independent of stresses (i.e. stiff clay), the resistance profile in the middle layer senses minimal influence from the stresses developed in the top layer. Therefore,
the resistance profiles in the middle stiff clay layer are the same from the partial and full LDFE analyses as \( d_s/D \geq 2 \).

### 6.5 CONCLUSION

The paper examines three methods to model CPT in three layered soils with soft-stiff-soft profile: small strain analysis, partial LDFE and full LDFE analysis. The soft layers are clay \((s_u = 20 \text{ kPa})\) and the stiff layer is either stiff clay \((s_u = 60 \text{ kPa})\) or medium dense sand \((I_D = 60\%)\). The efficiency of the partial LDFE analysis is investigated to save computational cost.

(a) The small strain analysis underestimates the cone resistance, except the cone is at the soil surface. The surface cone can reach its ultimate resistance within the small strain range. However, the pre-embedded cone never reaches the ultimate resistance up to \( d/D = 0.3 \). The results confirm the necessity of large deformation analysis to model the CPT problem.

(b) In order to save computation cost (i.e. time), the partial LDFE analysis can capture the cone resistance profile in the middle layer and below, as long as the cone is pre-embedded with enough distance to the middle stiff layer. For the soft-stiff-soft clay, \( d_s/D \geq 2 \) is required; and for the clay-sand-clay soil, \( d_s/D \geq 10 \) is essential for medium dense sand \((I_D = 60\%)\). \( d_s \) is the pre-embedded distance to the middle stiff layer.

(c) Based on the numerical results for a cone in soft-stiff-soft clay and clay-sand-clay soils, the influence of pre-embedment to the cone resistance profile is relatively small if the middle layer is stiff clay as its strength is not related to the soil stress level. However, the influence of pre-embedment is more significant if the middle layer is sand as its strength is dependent on the soil stress level.

This is a preliminary study on the effective partial LDFE analysis of CPT in soft-stiff-soft soil profiles. There is only one soil strength combination considered in each profile. More study is needed to provide more concrete suggestions with different soil strength combinations.
6.6 REFERENCES


Figure 6-1. Schematic plot of CPT in soft-stiff-soft soil

Figure 6-2. Load-displacement results of small strain analysis of CPT in soft-stiff-soft clay: (a) net resistance, $q_c$, (b) resistance ratio, $q_c/s_u$. (FL: failure load)
Figure 6-3. Comparison of cone resistance profiles by small strain and LDFE analyses for CPT in soft-stiff-soft clays (C-C: clay-clay interface; SS: small strain analysis)

Figure 6-4. Load-displacement results of small strain FE analysis in clay-sand-clay soil
Figure 6-5. Comparison of cone resistance profiles by small strain and LDFE analyses in clay-sand-clay soils (C-S: clay-sand interface; S-C: sand-clay interface)
Figure 6-6. Comparison between the results of partial and full LDFE analysis for CPT in soft-stiff-soft clays
Figure 6-7. Mean stress development for partial ($d_{pre}/D = 15$) and full ($d_{pre}/D = 0$) LDFE analysis of CPT in soft-stiff-soft clay
Figure 6-8. Comparison between the results of partial and full LDFE analysis for CPT in clay-sand-clay soils
Figure 6-9. Mean effective stress development for partial (d_{pre}/D = 18) and full (d_{pre}/D = 0) LDFE analysis of CPT in clay-sand-clay soils
Table 6-1. Summary of parametric study plan with cone pre-embedment depth (d_{pre})

<table>
<thead>
<tr>
<th>Analysis stream</th>
<th>Pre-embedded depth, d_{pre}/D</th>
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<tr>
<td>Small strain analysis</td>
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</tr>
<tr>
<td>Partial LDFE analysis</td>
<td>10, 15, 18, 20</td>
</tr>
<tr>
<td>Full LDFE analysis</td>
<td>0 (starting from the soil surface)</td>
</tr>
</tbody>
</table>
7 CONCLUSION

7.1 SUMMARY

This thesis reported a study of cone penetrations into layered soils using large deformation finite element (LDFE) analysis. The cones included both mini-cones used in centrifuge tests and the standard cone used in field tests. The layered soils included two to three-layer soil profiles involving both sand and clay, where sand was stress-dependent material under drained conditions and clay was modeled as a stress-independent material under undrained conditions. The Critical State Mohr-Coulomb (CSMC) model was deployed for sands in Chapter 3 to 6. The Tresca failure criterion was deployed for clays in Chapters 2 to 6.

This study has contributed to the advanced understanding of the cone penetration problem in layered soils aligned with the three objectives being laid out in Chapter 1. Figure 7-1 summarizes the contributions of this study with their coverage of soil strata and the corresponding cones.

7.2 CONTRIBUTION I

The thesis examined five different soil strata, as shown in Figure 7-1. For the first four strata (a-stiff over soft clay; b- sand over clay; c- a thin sand layer embedded in uniform clay; d- a thin sand layer sandwiched between different clays), formulas have been proposed to interpret the engineering properties of the soil in each layer and to identify the locations of the layer interfaces based on the measured cone resistance profiles, as included in the Chapters 2 to 5. For the strata e (soft-stiff-soft clay), interpretation formula has been proposed in other studies (Ma et al., 2016, Ma et al., 2017), as the objective of this study was to examine the efficiency of modelling methods.

All the proposed formulas can be illustrated based on standardized cone resistance profiles, as shown in Figure 7-2, where the horizontal axis is the net cone resistance (measured cone resistance excluding soil overburden pressure), and the vertical axis is the normalized penetration depth by
the cone diameter. The penetration depth is the depth measured from the cone shoulder (set as reference point) to the initial soil surface.

The two-layer soils (a- stiff over soft clay; b- sand over clay) are for miniature cones used in centrifuge tests. As the gravity level in centrifuge tests varies and can be up to 200g, the modelled cone size in the numerical study varies accordingly. The proposed interpretation formulas are independent of the cone prototype size for CPT in layered clays, which are a stress-independent material. However, the cone size does influence the interpretation formulas for CPT in layered soils, where sand is involved.

The three-layer soils are for the standard cone used in the field. The cone diameter is 0.0357 m, resulting in a 10 cm\(^2\) cone basement area, and this is consistent for all the analyses (c- a thin sand layer embedded in uniform clay; d- a thin sand layer sandwiched by different clays), Table 7-1 summarizes the proposed formulas by using the parameters depicted in Figure 7-2, to interpret soil strength and to identify layer interfaces. Soil strength parameters include the undrained shear strength \(s_u\) for clay and the relative density \(I_D\) for sand. The layer interface is identified through the kink points in the cone resistance profiles. The study assumed the top or bottom clay layers were thick enough to allow the cone resistance to reach its ultimate values, so the undrained shear strength of clay can be interpreted by the existing value of cone bear factor \(N_b = 11.5\).

It is worthwhile to note that the proposed formulas applies to thin layered soils, where the cone resistance could not be fully mobilized in the layer and cone senses impact from adjacent layers. When the layer is thick enough, the cone resistance will be fully mobilized and the fully mobilized cone resistance profile can be interpreted by conventional interpretation methods for single soil layers. For uniform layered clays, a constant net cone resistance with penetration depth can be deemed as ultimate resistance (fully mobilized resistance); while for sand layer, a constant
normalized cone resistance with penetration depth can be used as ultimate resistance (fully mobilized resistance).

7.3 CONTRIBUTION II

Over all the cases in this study, some typical soil flow mechanisms can be observed. They include (i) shallow flow mechanism; (ii) cavity expansion flow mechanism; (iii) flow towards lower soft layer; (iv) squeezing mechanism and (v) layer interface bending. The flow mechanisms can be linked to the characteristics of resistance profiles, including: (a) peak resistance in the stiff layers; (b) stable resistance in soft uniform clay layers; (c) kinks registered before and after the stiff layer.

The shallow flow mechanism, where the soil around the cone moves extensively with upward movement around the cone shaft to form soil heave, is always observed when the cone penetrates from the soil surface until either reaching deeply enough to form a localized flow mechanism (i.e. cavity expansion) or sensing a new soft layer underneath (i.e. flow towards lower soft layer). It is reflected as a nonlinear resistance increase in the resistance profiles initially until either reaching a constant ultimate resistance in a thick clay layer or a peak resistance in a thin stiff layer. The former is observed of a CPT in clay-sand-clay soils, where the top clay layer is thick, and the latter is observed of a CPT in stiff over soft soil.

The cavity expansion flow mechanism is observed when the cone penetrates deeply enough (more than 12D in uniform clay) and does not sense the influence from adjacent layers. The flow mechanism starts to be localized around the cone without reaching the soil surface for a top layer soil. Only limited soils around the cone moves vertically downward and laterally away from the cone. It is reflected as the ultimate constant resistance in uniform clay and linear resistance increase in sand in the cone resistance profiles.

Soil flow towards lower soft layer is observed when the cone senses a soft layer beneath the current soil layer. Soil vertical downwards displacement dominates. This soil flow mechanism starts when
the cone resistance in the stiff layer reaches its peak. With further penetration of the cone, shear planes form with the cone approaching the soft layer. At the same time, the high stresses around the cone tip in the current stiff layer are released gradually, resulting in cone resistance reduction to the ultimate resistance in soft layers. The soil flow towards lower soft layer happens in all the stiff layers, including stiff clay and sand layers, when the cone senses the underlying soft clay.

Soil squeezing is observed when the cone approaches a stiff layer from a soft clay. The soil is pushed laterally away from the cone face when the cone tip touches the layer interface, where soil horizontal displacements dominate. The soil squeezing mechanism induces a kink point of the cone resistance profile in the soft layer. With further penetration of the cone, the cone resistance increases sharply. The soft soil around the cone face keeps squeezing away until the cone shoulder passes through the layer interface. As the soil squeezing mechanism induces minimal deformation of the layer interface (i.e. soft-stiff interface), the kink distance (the distance measured from the kink to the original layer interface location) is close to the height of the cone face (i.e. \( \sim 0.87D \)).

The layer interface bending is observed when the cone penetrates from stiff to soft layers. The stiff-soft interface could sag into the soft layer with more than 1D. This is largely due to the soil mechanisms of flow towards lower soft layer, where downwards vertical displacements are dominant. This happens after the peak resistance is obtained in the stiff layer. The dominant downwards displacements in the stiff layer deforms the stiff-soft layer interface as sagging. A large sagging of the layer interface from its initial location can delay the cone passing through the interface, hence entering the lower soft layer. Therefore, the kink distance (i.e. the distance from the point when the cone resistance reaches its stable resistance in the soft layer to the initial layer interface location) become larger. The magnitude of the interface sagging (or the deformation of the interface) is a function of the stiffness ratio (i.e. stiffness of the stiff layer to the stiffness of the soft layer) between the adjacent layers. The higher the stiffness ratio, the more deformation (or more sagging) of the interface occurs.
When a thin sand layer is sandwiched in two different uniform clay layers, which is either stiff-sand-soft clay or soft-sand-stiff clay, a systematic soil failure is observed where all three soil layers are involved. For a cone penetration in stiff-sand-soft clay, the impact of the bottom soft clay can be sensed even when the cone is in the top stiff clay. This results in a cone resistance reduction in the top stiff clay after the ultimate stable resistance is reached. The less support from the bottom soft clay layer makes the thin sand layer behave like a beam, which is the sand layer sagging as the cone penetrates through it. Therefore, the mechanism of soil flow towards lower soft layer is observed in the very early stage of cone entering the sand layer, which induces the cone peak resistance near the top layer interface (i.e. clay-sand interface).

However, for a cone penetration in the soft-sand-stiff clay, the bottom stiff clay provides more support to the sand layer. Hence the sand layer exhibits much less bending and the mechanism of soil flow towards lower soft layer is observed in the later stage of cone entering the sand layer. Therefore, the cone peak resistance in sand appears closer to the lower interface (i.e. sand-clay interface).

The systematic soil flow mechanism become more obvious when the sand relative density ($I_D$) is very high. For the sand layer of $I_D = 30\%$ and $60\%$, the sand layer sags when the cone penetrates through it. The cone peak resistance in sand increases with increasing $I_D$. However, when the sand layer become very stiff as $I_D = 90\%$, a punching failure of the sand layer is observed. The punching failure makes the higher stress developed in sand around the cone can be released quickly. Hence the cone peak resistance in a sand of $I_D = 90\%$ lower than that of $I_D = 60\%$.

### 7.4 CONTRIBUTION III

As LDPE analysis for a deep penetration analysis of a cone in layered soils can be very time consuming, two means, which can save computational cost were examined to model cone resistance profiles in layered soils. The two means are: (i) pre-embedded small strain analysis; and (ii) pre-embedded LDPE analysis. The two means were applied to two soil strata profiles: (a) soft-
stiff-soft clay; and (b) clay-sand-clay. All results were benchmarked by the resistance profiles when LDFE analysis starts from the soil surface. The following conclusions resulted.

The pre-embedded small strain analysis is proved as always underestimating the cone resistance except when the cone is pre-embedded at the soil surface. The cavity expansion failure mode, when cone penetrates deeply, cannot be captured by the small strain analysis. In stress dependent materials, the analysis cannot reach a genuine failure load in the sense of displacement increment under the constant loading. It confirms that the LDFE analysis is necessary to model the soil response to cone penetration.

The pre-embedded LDFE analysis, which means to pre-embed the cone at certain distance to the stiff layer rather than starting from the soil surface, was examined to compute the cone resistance in the stiff layer embedded in soft clay. The resistance in the stiff layer can be correctly captured in two conditions: (1) when the cone is pre-embedded 2D above the stiff clay layer; (2) when the cone is pre-embedded 10D above the stiff sand layer. The latter long pre-embedment distance is required to mobilize the same stress field in the stiff sand layer as by full LDFE analysis. Based on the results, 10D is suggested as the minimum distance for medium dense sand ($I_D = 60\%$). When it aims to model the resistance profile in the embedded stiff layer, the pre-embedded LDFE analysis can be considered as one way to save computational cost compared with always starting the cone from the soil surface.

### 7.5 FUTURE WORK RECOMMENDATIONS

The project has extended the understanding of CPT in layered soil using LDFE analysis. The sand layer was modeled by an advanced soil model (critical state Mohr Coulomb – CSMC model) to capture its stress-dependent behaviors. The model was calibrated for Toyoura sand and UWA super fine silica sand. Guidelines on interpreting the CPT data in layered soil have been proposed for several soil strata. To expand the knowledge generated from this study, the following future work is recommended:
• When sands of interest are different from Toyoura sand and UWA super fine silica sand, the sand model needs to be calibrated for the corresponding sands. The proposed interpretation formulas in the study might need to be adjusted for different sands.

• The clay layers were modelled with a uniform shear strength in this study. A simple Tresca model was deployed to model its undrained behavior. In the future, the clay layers can be modelled as non-homogenous, with the undrained shear strength increasing linearly with depth. A lower value of E/su could also be deployed in the future analysis to cover softer clays. Additionally, advanced constitutive models covering anisotropy, strain rate-dependence and strain-softening could be used to study their effects on CPT in offshore/soft clay. However, since the FE analysis results of probe penetration problems by a simple Tresca model agree well with plasticity solutions, and the effect of anisotropy was less than 5% (Randolph and Anderson, 2006), the LDFE results in this study should present reasonable soil responses.

• The cone interface in this study was set as fully smooth. The cone roughness can be included in the future analysis to consider its effects on cone penetration resistance profiles.

• The thickness of the top clay layer was 20D for soil strata (d), where a thin sand layer is sandwiched by different clays. The top layer thickness can be varied to study its impact on the resistance profiles.

• The sand based LDFE modelling method could be applied to other penetration problems (e.g. a spudcan penetrating in layered soil), to further verify and extend the method.

• This work studied the cone penetrometer in clay under fully undrained condition and in sand under fully drained condition, hence pore water pressure generation is not considered. The excessive pore pressure generation and dissipation effects in layered soils could be studied in the future.
7.6 REFERENCE


Conclusion

Figure 7-1. Summary of contributions and their corresponding soil strata and cone
Figure 7-2. Typical cone resistance profile of (a) two-layer soil; (b) three-layer soil
Table 7-1. Summary of the equations to interpret the soil parameters

<table>
<thead>
<tr>
<th>Group</th>
<th>Soil strata</th>
<th>Location of the layer interfaces</th>
<th>Engineering properties of the soils</th>
</tr>
</thead>
<tbody>
<tr>
<td>Two-layer soils</td>
<td>Stiff over soft clay</td>
<td>• A pre-defined critical height ($H_c$) needs to be calculated: $\frac{H_c}{D} = 0.22 \ln \frac{q_p}{q_{cly}} + 8.31$ , Equation 7-1</td>
<td>• For the top stiff clay layer, the bearing factor ($N_{bp}$) is defined as: $N_{bp} = \frac{q_p}{s_ut}$ Equation 7-4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• then, $\left{ \begin{array}{l} \frac{d_k}{D} = 0.42 \ln \frac{q_p}{q_{cly}} + 0.19 \ln \frac{H_{nt}}{D} + 0.2, \quad H_{nt} \leq H_c \ \frac{d_k}{D} = 0.22 \ln \frac{q_p}{q_{cly}} + 0.81, \quad H_{nt} &gt; H_c \end{array} \right.$ Equation 7-2</td>
<td>• $N_{bp} = -0.6 \ln \left( \frac{q_p}{q_{cly}} \right) + 2.8 \ln \left( \frac{H}{D} \right) + 5.44$ Equation 7-5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Finally, $H = H_{nt} - d_k$ Equation 7-3</td>
<td>• For the bottom soft clay, the bearing factor ($N_b$) is 11.5.</td>
</tr>
<tr>
<td></td>
<td>Sand over clay</td>
<td>• $H = \frac{d_p}{D} = 0.11 + 0.746^{s1}$ Equation 7-6</td>
<td>• For the top sand layer, $I_d(%) = 23 \left( \frac{q_p}{p_0} \right)^{2.3} \left( \frac{P_0}{\sigma_{p0} S_{u0.994}} \right)$ Equation 7-7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>where $\sigma_{m^<em>}$ is the initial averaging effective stress in the sand layer: $\sigma_{m^</em>} = \frac{H}{2} \gamma'$, $\gamma'$ is the effective unit weight of the sand. where $p_0$ is the atmosphere pressure as the reference pressure. C is a coefficient changing with the sand layer thickness (H), as shown in Figure 7-3 (a).</td>
<td>• For the bottom soft clay, the bearing factor ($N_b$) is 11.5.</td>
</tr>
</tbody>
</table>
### Three-layer soils

A thin sand layer embedded in uniform clay

- For the top kink distance \(^*2\):
  \[
  \frac{d_{kt}}{D} = -0.01 \frac{H_t}{D} + 0.8, \quad \frac{H_t}{D} < 15
  \]
  \[
  \frac{d_{kt}}{D} = 0.65, \quad \frac{H_t}{D} \geq 15
  \]
  \[\text{Equation 7-8}\]
  \[
  \begin{align*}
  &\text{For the bottom kink distance \(^*3\):} \\
  &\frac{d_{kb}}{D} = 0.03 \frac{H_t}{D} + 0.75, \quad \frac{H_t}{D} < 15 \\
  &\frac{d_{kb}}{D} = 1.2, \quad \frac{H_t}{D} \geq 15
  \end{align*}
  \]
  \[\text{Equation 7-9}\]
  \[
  \text{Finally, } H = H_{tl} - d_{kt} - d_{kb}
  \]
  \[\text{Equation 7-10}\]

A thin sand layer sandwiched by different clays

- Same as the sand layer embedded in uniform clay.

For the sand layer:

\[
I_D(\%) = \frac{36 \left( \frac{d_p}{D} \right) \left( \frac{d_p}{p_a} \right)}{\left( \frac{S_u}{\gamma D} \right)^{0.85} \left( \frac{H_t}{D} \right)^\beta} - 18
\]

where \(\gamma\) is the unit weight of the clay. \(\beta\) is a coefficient varying with top layer thickness, as shown in Figure 7-3 (b).

For the top clay,

\[
N_b = \begin{cases} 
4.45 - 0.114 \left( \frac{d_t}{D} \right)^{1.5} + 3.31 \left( \frac{d_t}{D} \right)^{0.5}, & \frac{H_t - d_{kt}}{D} \leq 11 \\
11.5, & \frac{H_t - d_{kt}}{D} > 11
\end{cases}
\]

For the bottom soft clay, the bearing factor \((N_b)\) is 11.5.

\[\text{Equation 7-12}\]

For the sand layer:

\[
I_D(\%) = 100 \frac{q_p \gamma^2 s_{ut}^2 s_{ub}^2 (H_t / 20D)^5 (\gamma_D)^2}{\left[ C_1 (\alpha s_{ut} + (1-\alpha)s_{ub}) - C_2 \right]^2 p_a^6}
\]

where \(\alpha\) is 0.3. For \(s_{ut}/s_{ub} > 1.0\), the factors are \(C_1 = 7269.7\) and \(C_2 = 215331\); and for \(s_{ut}/s_{ub} < 1.0\), \(C_1 = 518.89\) and \(C_2 = 1640\).

\[\text{Equation 7-13}\]
For the top clay, the bearing factor ($N_b$) is applied to the ultimate resistance, which is reached after 12D.

For the bottom soft clay, the bearing factor ($N_b$) is 11.5.

*** It requires iteration process to solve these three equations, as the unknown variables are on both sides of the equation.

- *1 - the total height measured from the soil surface to the kink point ($H_{in}$) is suggested to start the iteration.
- *2 and *3 - the measured depth from the soil surface to the top kink point is suggested for the value of $H/D$ to start the iteration.

Figure 7-3 (a) Coefficient $C$ in the interpretation formula for the relative density of the sand layer over clay (Equation 7-7); (b) Coefficient $\beta$ in the interpretation formula for the relative density of the sand layer embedded in clay (Equation 7-11).